Design of Edge Beams

EZDIN DURAN
Design of edge beams

Ezdin Duran

June 2014
TRITA -BKN. Master Thesis 427, 2014
ISSN 1103-4297
ISRN KTH/BKN/EX-427-SE
© Ezdin Duran, 2014
KTH Royal Institute of Technology
Department of Civil and Architectural Engineering
Division of Structural Engineering and Bridges
Stockholm, Sweden, 2014
Preface

I would like to give thanks to my supervisors Prof. Håkan Sundquist, Ulf Sandelius from Broteknik Ulf Sandelius AB and PhD student José Javier Veganzones Muñoz as well as my examiner Adj. Prof. Lars Pettersson for their guidance and help during my work with this master thesis. It has been an honor to get the chance to work alongside each and one of you.

Special thanks to master student Martti Kelindeman whose master thesis made it possible for me to accompany on site visits.

I would also like to give thanks to the Structural Engineering and Bridge Division at KTH for providing working space during this thesis.

Ever so thankful to COWI, ELU, Skanska and NCC for gathering of information for this thesis as well as organizing site visits to different bridges.

Finally I would like to thank my family and friends for supporting me during this period of 5 years.
Abstract

The purpose of the edge beam is to support the railing and the pavement, function as part of the drainage system and in the case it is integrated into the bridge deck it can serve to distribute concentrated loads. It is located in road environment and therefore exposed to water and salt with chlorides as well as subject to impacts during accidents. It deteriorates in a greater pace than the rest of the bridge and therefore has a shorter lifespan than the bridge in full. A deteriorated edge beam put the safety of the bridge users in jeopardize and increases the need of maintenance, repair and replacement work. These activities affect the surrounding traffic flow due to reduced speed limits as well as closure of traffic lanes.

A literature study has been performed to get an understanding of how edge beams are designed and constructed. A great part of this was done by examining codes and regulations. By meeting engineers from different building companies it has been possible to obtain a picture of how it is done in real life and how the path to the final design looks like. Building site visits were carried out to see the process from design to construction i.e. how it is applied in real life. A design study was performed, including a check of crack width in an integrated edge beam over a support, height of bridge deck when a pre-fabricated (brokappa) is used and a comparison in the magnitude of the clamping moment in a steel-concrete bridge with and without an edge beam. All proposals are presented by the Edge Beam Group (EBG, in Swedish, Kantbalksgrupper), which is composed of experienced engineers that works within the frame of the project social optimal edge beam systems governed by the Swedish Transport Administration.

The literature research showed that even if the edge beam is prone to deteriorate its lifespan does not have to be governed by its condition. Planned expansion of bridge width and maintenance strategies including the replacement of waterproofing layer could also be a reason for replacement in some cases.

A significant increase of reinforcement in the edge beam and top part of the bridge deck over support is needed to obtain an acceptable crack width of 0.15mm. This would however aggravate the casting phase. The use of a pre-fabricated edge beam result in an increase of the bridge deck height. A solution could be to strengthen the anchoring capacity but this could in turn give an over reinforced structure. When it comes to the clamping moment in a steel-concrete composite bridge the integrated edge beam leads to a better distribution of the traffic load. On the other hand, due to the higher dead weight, a bridge deck without an edge beam would result in a lower total moment in the cantilever.

Keywords: Edge beam, deterioration, design, construction, codes and regulations, building companies, integrated edge beam, pre-fabricated edge beam, without an edge beam, edge beam group, crack width, clamping moment
Sammanfattning


För att få en förståelse för hur kantbalken dimensioneras och konstrueras har en litteraturstudie och granskning av regelverk som härrör till ämnet genomförts. Genom att träffa ingenjörer från olika byggföretag har det varit möjligt att klargöra hur dessa dimensioneras och hur vägen till slutgiltig design ser ut. Arbetsplatsbesök gjorde det möjligt att se utvecklingen från dimensionering till konstruktion och arbetet däremellan. Som sista del av detta arbete utfördes dessutom en undersökning av tänkbara dimensioneringsfrågor så som sprickvidds beräkningar i en integrerad kantbalk över stöd, höjd av brobaneplatta då en pre-fabricerade kantbalk används och en jämförelse av moment i en konsol med och utan en kantbalk. Förslag på kantbalksutförningar har lagts fram av kantbalksgruppen, en grupp specialister som arbetar inom trafikverkets projekt som handlar om att utforma optimala kantbalkssystem.

Resultatet av litteraturstudien visar att kantbalken är särskilt utsatt, men det är svårt att säga att de byts ut enbart för att kantbalken är i dåligt skick. I vissa fall kan den bytta ut då bron är i behov av en breddning eller då isoleringen är i dåligt skick.

Undersökningen av dimensioneringsfrågor bevisade att det praktiskt taget är för svårt att hålla ner sprickvidden för integrerade kantbalkar över stöd, ökningen av mängden armring hade försvärat gjutningen av kantbalk och brobaneplatta avsevärt. Att använda en pre-fabricerad kantbalk resultera i att brobaneplattan behöver tjockas till eller att förankringsstyrkan ökas vilket i sin tur kan leda till att kantbalken blir överarmerad. Beslutet att inte använda en kantbalk kan resultera i att en mindre inspänningsmoment vid stöd för mindre broar erhålls.

Nyckelord: Kantbalk, regelverk, byggföretag, dimensionering, konstruktion, integrerad kantbalk, pre-fabricerad kantbalk, utan egentlig kantbalk, kantbalksgruppen, sprickvidd, inspänningsmoment
Contents

Preface ....................................................................................................................................................... i
Abstract ...................................................................................................................................................... iii
Sammanfattning ........................................................................................................................................ iv
Nomenclature ............................................................................................................................................... ix

1 Introduction ............................................................................................................................................... 1
  1.1 General background .......................................................................................................................... 1
  1.2 Aim and scope ..................................................................................................................................... 2
  1.3 Methodology ....................................................................................................................................... 3
    1.3.1 Literature study ........................................................................................................................... 3
    1.3.2 Building site visits ....................................................................................................................... 3
    1.3.3 Interviews .................................................................................................................................... 3
    1.3.4 Design study .................................................................................................................................. 5
  1.4 Assumptions and limitations ............................................................................................................ 5

2 The edge beam ........................................................................................................................................ 7
  2.1 Definition of concrete bridge deck .................................................................................................... 7
  2.2 Bridge edge beam system .................................................................................................................. 7
    2.2.1 Definition of bridge edge beam system (BEBS) ....................................................................... 7
    2.2.2 Definition of edge beam ........................................................................................................... 8
    2.2.3 Railings ....................................................................................................................................... 11

3 Durability of edge beams ...................................................................................................................... 13
  3.1 Deterioration during construction phase ........................................................................................... 15
    3.1.1 Plastic shrinkage cracks ............................................................................................................. 15
    3.1.2 Thermal contraction cracks ....................................................................................................... 16
  3.2 Service life related deterioration ...................................................................................................... 16
    3.2.1 Shrinkage ..................................................................................................................................... 16
    3.2.2 Flexural (bending) cracks ......................................................................................................... 17
    3.2.3 Carbonation ............................................................................................................................... 17
    3.2.4 Chloride intrusion ....................................................................................................................... 17
    3.2.5 Frost wedging ............................................................................................................................. 18
3.2.6 Deterioration in real life

3.3 Preventive maintenance

3.3.1 Impregnation

3.3.2 Cathodic protection of reinforcement

3.3.3 Stainless steel reinforcement

3.4 Corrective maintenance

3.4.1 Concrete repair

3.4.2 Crack injection

3.4.3 Replacement

4 Building site visits

4.1 Case study 1

4.2 Case study 2

4.3 Case study 3

5 Design study

5.1 Controls for an integrated edge beam

5.2 Cracks over supports

5.3 Anchorage of an pre-fabricated edge beam

5.4 Distribution of point load

6 Discussion

6.1 Codes and standards

6.2 Bridge cases

6.2.1 Implementation of edge beam elements

6.2.2 H4-railing
6.2.3 Landskapsbron ................................................................. 42
6.3 Cracks over support ............................................................. 43
6.4 Anchorage of pre-fabricated edge beam .................................... 43
6.5 Clamping moment in a cantilever ............................................ 43

7 Conclusions .................................................................................. 45
7.1 Codes and standards ............................................................... 45
7.2 Bridge cases ........................................................................... 45
  7.2.1 Askersund ......................................................................... 45
  7.2.2 Rotebro E4 .................................................................... 45
  7.2.3 Landskapsbron ............................................................... 46
7.3 Design study ........................................................................... 46
  7.3.1 Cracks over support .......................................................... 46
  7.3.2 Anchorage of pre-fabricated edge beam ......................... 46
  7.3.3 Clamping moment in a cantilever .................................... 47
7.4 Further research ...................................................................... 47

8 Bibliography .................................................................................. 49

Appendix A: Questionnaire ............................................................. 52
Appendix B: Landskapsbron ............................................................. 59
Appendix C: Rotebro E4 ................................................................. 66
Appendix D: Drawings design study ............................................... 71
Appendix E: Calculations ............................................................... 77
# Nomenclature

## Greek Letters

<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>γs</td>
<td>Carrying capacity of steel</td>
<td>-</td>
</tr>
<tr>
<td>γc</td>
<td>Carrying capacity for concrete</td>
<td>-</td>
</tr>
<tr>
<td>φ\text{bolt}</td>
<td>Diameter of bolt</td>
<td>mm</td>
</tr>
<tr>
<td>φ\text{eb}</td>
<td>Diameter of longitudinal reinforcement in edge beam</td>
<td>mm</td>
</tr>
<tr>
<td>φ\text{bd}</td>
<td>Diameter of longitudinal reinforcement in bridge deck</td>
<td>mm</td>
</tr>
<tr>
<td>φ\text{shackle}</td>
<td>Diameter of shackle reinforcement in edge beam</td>
<td>mm</td>
</tr>
<tr>
<td>σ\text{sd}</td>
<td>Design stress</td>
<td>Pa</td>
</tr>
<tr>
<td>η</td>
<td>Degree of capacity utilization</td>
<td>-</td>
</tr>
<tr>
<td>η1</td>
<td>Adhesion and bar location coefficient</td>
<td>-</td>
</tr>
<tr>
<td>η2</td>
<td>Coefficient related to bar diameter</td>
<td>-</td>
</tr>
<tr>
<td>γ\text{concrete}</td>
<td>Density of concrete</td>
<td>N/m³</td>
</tr>
<tr>
<td>ε\text{cu}</td>
<td>Ultimate compressive strain of concrete</td>
<td>-</td>
</tr>
<tr>
<td>εs</td>
<td>Strain in steel</td>
<td>-</td>
</tr>
<tr>
<td>ρ\text{p.eff}</td>
<td>Percentage of reinforcement in concrete area</td>
<td>-</td>
</tr>
</tbody>
</table>

## Roman Letters

<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>A\text{c}</td>
<td>Concrete area</td>
<td>m²</td>
</tr>
<tr>
<td>A\text{cc}</td>
<td>Area of concrete compression part</td>
<td>mm²</td>
</tr>
<tr>
<td>A\text{eb}</td>
<td>Edge beam area</td>
<td>m²</td>
</tr>
<tr>
<td>A\text{s}</td>
<td>Steel area</td>
<td>mm²</td>
</tr>
<tr>
<td>A\text{s.uls}</td>
<td>Amount of reinforcement in ultimate limit state</td>
<td>mm²</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Unit</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td>------</td>
</tr>
<tr>
<td>$A_{s,sls}$</td>
<td>Amount of reinforcement in serviceability limit state</td>
<td>mm$^2$</td>
</tr>
<tr>
<td>a</td>
<td>Internal lever arm of bolt group</td>
<td>mm</td>
</tr>
<tr>
<td>$c_c$</td>
<td>Concrete covering surface</td>
<td>mm</td>
</tr>
<tr>
<td>$c.c$</td>
<td>Distance between railings</td>
<td>m</td>
</tr>
<tr>
<td>d</td>
<td>Effective height</td>
<td>mm</td>
</tr>
<tr>
<td>$E_{cm}$</td>
<td>Mean modulus of elasticity for concrete</td>
<td>Pa</td>
</tr>
<tr>
<td>$E_a$</td>
<td>Modulus of elasticity for reinforcement</td>
<td>Pa</td>
</tr>
<tr>
<td>$e_h$</td>
<td>Eccentricity from load to top of the coating</td>
<td>m</td>
</tr>
<tr>
<td>F</td>
<td>Traffic load</td>
<td>N</td>
</tr>
<tr>
<td>$F_c$</td>
<td>Compressive concrete force</td>
<td>N</td>
</tr>
<tr>
<td>$F_H$</td>
<td>Vehicle impact load</td>
<td>N</td>
</tr>
<tr>
<td>$F_t$</td>
<td>Tensional reinforcement force</td>
<td>N</td>
</tr>
<tr>
<td>$f_{ck}$</td>
<td>Characteristic compressive concrete strength</td>
<td>Pa</td>
</tr>
<tr>
<td>$f_{ctd}$</td>
<td>Design tensile strength of concrete</td>
<td>Pa</td>
</tr>
<tr>
<td>$f_{ctm}$</td>
<td>Mean value of tensile concrete strength</td>
<td>Pa</td>
</tr>
<tr>
<td>$f_{uk}$</td>
<td>Characteristic yield limit of railing</td>
<td>Pa</td>
</tr>
<tr>
<td>$f_{yk}$</td>
<td>Characteristic yield limit of steel</td>
<td>Pa</td>
</tr>
<tr>
<td>$h_{cc}$</td>
<td>Height of concrete compression part</td>
<td>mm</td>
</tr>
<tr>
<td>$h_{bd}$</td>
<td>Height of bridge deck</td>
<td>mm</td>
</tr>
<tr>
<td>$h_{eb}$</td>
<td>Height of edge beam</td>
<td>mm</td>
</tr>
<tr>
<td>I</td>
<td>Moment of inertia</td>
<td>m$^4$</td>
</tr>
<tr>
<td>i</td>
<td>Moment of inertia per meter</td>
<td>m$^4$/m</td>
</tr>
<tr>
<td>$l_{b,b}$</td>
<td>Anchorage length of bolt</td>
<td>mm</td>
</tr>
<tr>
<td>$l_{bd}$</td>
<td>Design anchorage length</td>
<td>mm</td>
</tr>
<tr>
<td>$l_{side}$</td>
<td>Side length of railing</td>
<td>mm</td>
</tr>
<tr>
<td>$M_r$</td>
<td>Moment resistance</td>
<td>Nm</td>
</tr>
</tbody>
</table>
M_s  Moment                      Nm
M_{sls}  Moment at serviceability limit state    Nm/m
M_{uls}  Moment at ultimate limit state         Nm/m
m_{ed}  Dimensioning moment in edge beam        Nm
n        Number of rebars (bolts)                -
n_{ed}   Dimensioning normal force in edge beam  N
q        Distributed load                       N/m
S_{r,max} Maximum crack spacing                 mm
T_{ed}   Design torsional moment                Nm
t_{eb}   Distance from coating to edge beam top  mm
t_{p}    Point of gravity                       mm
V_{ed}   Design shear force                     N
w_{eb}   Width of edge beam                     mm
w_k     Crack width                            mm
z        Plastic bending resistance              m^3

Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ADT</td>
<td>Average Daily Traffic</td>
</tr>
<tr>
<td>BEBS</td>
<td>Bridge Edge Beam System</td>
</tr>
<tr>
<td>E4</td>
<td>European road 4</td>
</tr>
<tr>
<td>E6</td>
<td>European road 6</td>
</tr>
<tr>
<td>EBG</td>
<td>Edge Beam Group</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite Element Modeling</td>
</tr>
<tr>
<td>LCC</td>
<td>Life Cycle Costing</td>
</tr>
<tr>
<td>LM1</td>
<td>Traffic Load Model 1</td>
</tr>
<tr>
<td>STA</td>
<td>Swedish Transport Administration</td>
</tr>
</tbody>
</table>
1 Introduction

1.1 General background

Sweden is among the countries with most amount of infrastructure per inhabitant. However the investments into this sector have decreased from back in the 1970s when they were among the countries with the highest investments into the infrastructure sector, up to now when they are below average (Blomqvist 2012).

The economy is very important for the wellbeing of a good infrastructure. It is the investments from the government and local authorities that leave space for development. These goes hand in hand, a reduced budget for this sector results in a poor infrastructure that are in need of more maintenance, a high maintenance need which in turn affects the national economy. A big part of the infrastructure in Sweden consist of bridges, a majority of these are made of concrete with integrated edge beams supporting the railings. The safety concerns are many and a reason for the need of inspections and maintenance work on a regular basis.

The picture of the situation today is that the edge beam is not of great importance to the designer and not focused upon in detail. Much of the standards and regulations from the past are still used when designing today. As proved in previous projects it is among the most deteriorated parts of the bridge (Racutanu 2001). A deteriorated edge beam decreases the functionality of the railing and puts the safety that it provides in jeopardize. It is one of the most exposed structures of the bridge and is vulnerable to water, salts, airborne pollutions, traffic loads and impacts from vehicles.

The bridge over Ångermanälven is a railway bridge with edge beams taking up to 40 percent of the cross sectional area of the bridge deck (see figure 1). This bridge is evidence on that the crack width limitations for the edge beams should be checked (Ansnaes 2012).
The Swedish Transport Administration (STA) has therefore provided KTH with a task to come up with a more robust edge beam(s) that is optimal for the society. The project reaches all the way up to senior level with one PhD student beneath them and two master students working with Life Cycle Costing (LCC) and Design questions regarding the edge beam. At senior level 24 proposals have been developed by the Edge Beam Group (EBG), a group of scientist and designers that are investigating if it is possible to develop and improve the edge beams used today. In this thesis the design part will be handled.

1.2 Aim and scope

The aims and scope of this work are:

- To find out how companies design their edge beams. How they work to come up with the final design for the edge beam and what kind of aspects along the way play a big role in the decision making process.
- To investigate how precise the codes and regulations are i.e. to see if possible problems lay within the codes and regulations, in this case the Swedish nation annex TRV(KR) Bro 2011.
- To see in practice how the edge beams are constructed and obtain knowledge about common problems that can occur during the construction phase. To obtain a wide view both short and long span bridges were examined as well as bridges with high and low ADT values.
- All investigations were performed on road bridges due to that they are exposed to deicing salts and vehicle loads.
- To investigate some design criteria’s of a number of designs provided by the EBG and to draw some general conclusions of these designs as:
  - If it is possible to keep the crack width beneath the limitation for an integrated edge beam over support.
  - If a comparison of clamping moment in a cantilever with and without an edge beam will lead to that it is good idea to exclude the use of an edge beam.
  - How well the pre-fabricated edge beam can withstand an impact and if extra measures are needed if it will be used.
1.3 Methodology

1.3.1 Literature study

The literature study has been a great part of this master thesis. First of all, general information about edge beams were collected to get a picture of the problems and how the situation is today, some of this information have been presented in chapter 2. Information about deterioration processes as well as repair and replacement works have been gathered and can be found in chapter 3. The codes and standards used in Europe, North America and Sweden were examined to understand what kind of regulations and rules that has to be followed when dimensioning the edge beams, this was fundamental to be able to carry out the design study. The information gathered during the literature study were used as a build up for the questionnaire and design study.

1.3.2 Building site visits

To get some practical knowledge about edge beams, building site visits have been carried out to NCC and Skanska sites during this period. The main goal was to see how the preparations, execution and finishing touch are carried out i.e. preparation of formwork and reinforcement, casting and hardening. Each bridge has a different design and execution process, making it possible to get a wider perspective of the edge beam in general. This gave the opportunity to see eventual drawbacks and advantages that comes with different types of designs.

1.3.3 Interviews

One part of the analysis is the interviews that were held with different companies, authorities and individuals both in Sweden and in Denmark. By preparing an questionnaire that were used for telephone and real life meetings, questions as how the companies work, how the edge beam is designed and why a special kind of edge beam design are chosen could be answered [see appendix A].

1.3.3.1 Semi structured interviews

In this master thesis interviews of the semi-structured kind have been performed to get the most out of the contacts.

- Unstructured ≈ observation
- Structured ≈ questionnaire

First step was to send out a questionnaire to the contacts so that they could prepare themselves for either a telephone or real life meeting that would take place in the near future. During these meetings the answers to the questions were discussed more thoroughly.

There is always a threat that the interviewers preconceived ideas and thoughts will result in that leading questions will be asked (Newton 2010). However by performing a majority of the
literature study before the interviews took place hopefully preconceived thoughts could be eliminated. The main goal was that each interview would be unique, in that sense that the interviewed would set the tone for each meeting, in that way developing an own coherence.

### 1.3.3.2 Companies

#### 1.3.3.2.1 COWI

COWI is a company that started out in Copenhagen for about 80 years ago but is today active in countries all over the world. They operate in Sweden and have a number of offices on different locations. COWI has been a part of many big bridge projects including the Oresund Bridge that Skanska was part of, a bridge between Denmark and Sweden. When it comes to the field of bridges COWI is a consulting company that offers services covering the whole lifecycle period of a bridge (COWI 2013).

![COWI](image)

**Figure 2** COWI-logo (COWI 2013)

#### 1.3.3.2.2 ELU

ELU has an experience of 40 years in the building business and are one of the leading ones in this field. With a majority of civil engineers in the company ELU has one of the most developed consulting departments within the fields of facilities in Sweden. At this department bridge projects are very common and the reason for this is due to the high competency in the field of bridges. ELU has the ability to be part of the planning all the way to building documents, inspections etc. (ELU 2014).

![ELU](image)

**Figure 3** ELU-logo (ELU 2014)

#### 1.3.3.2.3 NCC

Nordic Construction Company or NCC as it is called, is one of the leading construction companies in Scandinavia and is mainly active in Scandinavia. NCC is a young company but has its history in two other companies named Armerad Betong Vägförbättringar (ABV) and Johnson Construction Company (JCC). NCC has a great knowledge in the field of bridges and often provides new solution for bridges as e.g. the maintenance free steel-concrete composite bridge for spans up to 100m (NCC 2014).
1.3.3.2.4 Skanska

Skanska with its 56,600 employees around the world started out in the early years of 1887 and is one of the largest construction companies in the world today (Skanska 2012). A big part of Skanska’s organisation is targeting buildings and infrastructure and have a long history of involvement in big bridge projects, as the Oresund Bridge for example. At Skanska they focus on building environmental friendly and offer total solution to entrepreneur executions.

1.3.4 Design study

A design study of Swedish edge beams has been carried out according to Eurocode and TRVK/R Bro 2011, with the help of calculations provided by the building companies. The first step was to perform local controls that are done when designing. From there on the crack width in edge beams over supports, anchoring of the pre-fabricated edge beams and clamping moment at the support with the edge beam as a force distributing part were analyzed. For the crack width control a real life bridge located in Kista, Stockholm was used and for the load distribution a real life E6 bridge close to Dynekilen, Högdal was used [see appendix D].

1.4 Assumptions and limitations

The assumption before the work had started was that building companies probably work in the same patterns as they have always done i.e. previous solutions that have been already proven to work well are used again. That the edge beam is neglected when dimensioning the bridge and that is why it is in need of continuous maintenance work.

When this kind of work is done i.e. a research that is based on meetings and interviews some limitations will always aggravate the work. Some of these are:

- The most optimal thing is that as many countries as possible with the same weather conditions as in Sweden are included. In this work only Denmark except for Sweden was included and only one company in Denmark. Countries as the United States, Canada, Scandinavia and countries in the north of Europe could also participate in this kind of study. This would give a wider perspective of edge beams.
- Handling with people there is always a different approach from person to person as well as company to company. The interest in this topic, drive and opportunity to be involved in this work is not the same for all parts involved.
In this master thesis a FEM-program was not used, only hand calculations were done. With a FEM-program the influence of the edge beam as a part of the bridge deck could possibly be investigated in a more general sense. In the end more parts could be included in the design study.

When it comes to the practical part of the edge beam a study concerning the influence of deterioration of edge beams was not carried out.
2 The edge beam

2.1 Definition of concrete bridge deck

The concrete deck is a vital part of the bridge superstructure. Its purpose is to work as a path for the traffic that travels across the bridge and transfer the loads coming from traffic to other load-carrying bridge parts. When talking about the bridge deck in this report it will be referring to the top part, this is illustrated in figure 6. The deck consist of the concrete slab, sealing, pavement, edge beam, railing, drainage and expansions joints (Faridoon et al 2011).

![Figure 6 Illustration of concrete bridge deck (Faridoon et al 2011)](image)

2.2 Bridge edge beam system

2.2.1 Definition of bridge edge beam system (BEBS)

The Bridge Edge Beam System is out of a safety concern one of the most important parts of the bridge. It has been proven by George Racutanu that the BEBS is among the parts of the bridge that are in most need of repair and replacement work (Racutanu 2001). The BEBS consist of the edge beam, waterproofing layer, railing and drainage system. The focus in this report will be on the design of the edge beam with the railing included. These two parts are considered to be in most need of repair and replacement work. The BEBS is illustrated in figure 7 without expansion joints.
2.2.2 Definition of edge beam

The edge beam is positioned longitudinally at the edge of the bridge deck. The main function of the edge beam is to work as support for the railing and the pavement and to be a part of the drainage system. Alongside these attributions the edge beam is seen as a stiffening structure of the bridge deck and can also be of a load-carrying or non-load-carrying type (Sundquist 2011). In the case when a load-carrying edge beam is used it will distribute concentrated loads derived from traffic. In turn when a non-load-carrying edge beam is used it is thought that it will not distribute the load. The four types of designs provided by the EBG are showed in the figures below, keep in mind that the scale may differ from type to type.

2.2.2.1 Integrated concrete edge beam

The integrated edge beam (integnerad kantbalk) is the most common type of edge beam used in Sweden, particularly the raised one that also works as a support for the wearing course (see figure 8). The integrated edge beam is cast together with the bridge deck leading to that it contributes to the stiffness and helps distributing loads acting on the cantilever.
2.2.2.2 Pre-fabricated concrete edge beam

The pre-fabricated concrete edge beam (pre-fabricerad kantbalk) has been known to be used in Sweden but not in many cases. It is used in some European countries e.g. Germany and Switzerland. It is not cast together with the bridge deck leading to that the joint between the edge beam and bridge deck is problematic out of a maintenance point of view. The benefit with this type of design is that it can be replaced very easy and without affecting the traffic flow (see figure 9). It does not contribute to the bridge decks total stiffness in the same sentence as the integrated edge beam. However in those cases when the edge beam element is pre-fabricated and cast together with the bridge deck it will contribute to the bridge decks total stiffness, this is the case for the bridge in Askersund.
2.2.2.3 Without a real edge beam

Without a real edge beam (utan egentlig kantbalk) is a type of solution where the railing is directly attached to the bridge deck (see figure 10). This type is probably not used in Sweden but it has been said that it is used in the United States. This is a prototype developed by the EBG and it is an interesting example that would decrease the extra costs of pouring an edge beam. Possible disadvantages with this type are the joint between the railing and bridge deck and how the solution for the isolation would look like.

![Figure 10 Without a real edge beam (EBG 2014)](image)

2.2.2.4 Steel edge beam

The steel edge beam (stål kantbalk) is a type of solution where the railing is attached to a steel plate as can be seen in figure 11. There is no facts for the time being saying that this type is used anywhere in the world. It is the most unheard-of design of the EBG presented prototypes. Possible disadvantages are that it will be too expensive, penetration of the joint between the steel and concrete and deterioration of exposed steel parts.
2.2.3 Railings

Railings are used to protect bridge user from falling off the bridge, both pedestrians and vehicles. Small vehicles as well as large ones travel along bridges and the purpose is to prevent vehicles from falling over the edge and in best way direct them back towards the road lane i.e. catch it as softly as possible. It should not harm the vehicle more than the damage obtained if it drove over the edge of the bridge (SVBRF 2014). Common types used in Sweden and other countries are guard rail, pipe rail and bar rail (see figure 12).

In later years the pressure on bridges has increased, this is a result of increasing weight and speed of vehicles. This in turn leads to that they have to be able to handle this kind of traffic. Recently a new type (H4-railing) that has been tested for an impact of over 30 tonnes has been constructed (Trafikverket 2013). The different classes that exist today are the H2-, H3- and H4-railing (see table 1). The most common type of these is the H2-railing.
**Table 1** Crash test of railings for vehicles (Trafikverket 2013)

<table>
<thead>
<tr>
<th>Railing</th>
<th>Weight [tonnes]</th>
<th>Speed [km/h]</th>
</tr>
</thead>
<tbody>
<tr>
<td>H2</td>
<td>13</td>
<td>70</td>
</tr>
<tr>
<td>H3</td>
<td>16</td>
<td>80</td>
</tr>
<tr>
<td>H4</td>
<td>38</td>
<td>65</td>
</tr>
</tbody>
</table>
3 Durability of edge beams

The most common edge beam used for concrete bridges in Sweden is the integrated concrete edge beam. New bridges are often designed for a service life between 80 and 120 years. The edge beam is a problematic part that requires a lot of maintenance work during the service life of the bridge. To make it clear in some cases even when the bridge has theoretical service life of 100 years, it could well surpass the presumed time if it is located in a non-aggressive environment (Ansell et al 2012).

Due to the harsh climate in Sweden, bridges are affected not only by human activity but also environmental actions. The edge beam is exposed to airborne pollution, waterborne pollution and loads from bridge users. Deterioration processes that are highlighted in this thesis are carbonation-, chloride intrusion and cracks, which are the main agents causing corrosion of reinforcement. Cracks expose the reinforcement and can be introduced during construction phases and service life of the bridge as a result of flexural moment, frost and thaw attacks and different types of shrinkage processes.

The staple diagram in figure 13 comes from Hans-Åke Mattsons report and shows that 135 edge beams were replaced and 125 edge beams were repaired in Mälardalen region during the period 1990-2005. For these bridges the average age for repair was lower than for replacement.

![Figure 13](image.png)

*Figure 13* Replaced and repaired edge beams in Mälardalen region during the period 1990-2005 (Mattsson 2008)
Out of the 135 replaced edge beams 30 were located on European roads and 105 on other roads. As can be seen in Figure 14 the average age is lower for the European road bridges (Mattsson 2008). Possible reasons for this is that these have a high ADT value, which in turn increase the safety precautions as well as that more de-icing salt are used compared to other bridges, increasing the speed of deterioration. Another reason could be that European roads are highly prioritized.

![Figure 14](image)

*Figure 14  Replaced edge beams located on European- and other roads (Mattsson 2008)*

In Racutanu’s report a number of Swedish bridges have been inspected and the outcome has been registered. To obtain good and fair result bridges along the same patch were inspected, it is thought that these often are built during the same period and designed with the same codes and regulations. It is also believed that these are exposed to approximately the same ADT value and the same amount of de-icing salts. In Figure 15 it is clear that the edge beam and rest of the BEBS parts are the most deteriorated parts (Racutanu 2001).

![Figure 15](image)

*Figure 15  Distribution of deterioration for 353 Swedish road bridges (Racutanu 2001)*

In the same report the distribution of the most deteriorated bridge parts were registered i.e. those structural parts that were considered to be inadequate at the time of inspections (see
The waterproofing was the most deteriorated part of the 353 bridges (Racutanu 2001). A rough interpretation of this is that when the waterproofing is in need of a replacement the edge beam could be replaced at the same time to reduce the impact on traffic. However there is still a large amount of edge beams that are considered to be inadequate and whether the waterproofing is replaced or not these are in need of a replacement. Also in a couple of years the edge beams that have been inspected and shown to be deteriorated will be considered inadequate.

3.1 Deterioration during construction phase

3.1.1 Plastic shrinkage cracks

Just after the concrete has been poured and not hardened yet it has plastic properties. During the upcoming hours cracks may occur in the edge beam. The concrete has low deformation resistance during the first couple of hours after pouring and a low capacity to take up loads (Ansell et al 2012). Cracks can therefore develop deep into the edge beam. When the concrete has been poured into the formwork and vibrated it will settle for the upcoming hours. The consistence of it will lead to that water move upwards during the bleeding phase and evaporate from the surface. If the water evaporates from the concrete surface faster than water can move up toward the surface a net loss of water will develop (see figure 17). The surface part will try to shrink because of the reduced volume from the net loss of water but will not be able to do so due to that underlying parts is still saturated. This will result in an incomplete internal restraint that will give rise to tensile stresses in the surface part. The low capacity of concrete to take up loads in plastic condition will lead to that cracks develop when it is exposed to tensile stresses (Concrete society 2014).
3.1.2 Thermal contraction cracks

When concrete hydrates heat is generated from the reactions that take place. During a couple of weeks after the concrete has been poured it is sensitive to thermal stresses. It expands during hydration and when it cools down the restraint in it will give rise to tensile stresses. These tensile stresses can be too great for the concrete that has not yet obtained full strength from the hydration (Concrete Society 2014).

Thermal contraction cracks can develop when the edge beam and bridge deck are cast on different occasions. To solve this problem pipelines with heated water can be used to heat up the bridge deck to the same level as the edge beam so that they can cool down in the same pace (Jyttner 2014).

When large edge beams are poured thermal contraction cracks can develop at the surface. The surface and core cools down in different pace. When the surface cools down and starts to contract but are restraint by the still heated core, contraction cracks can develop (Pettersson 2014).

3.2 Service life related deterioration

3.2.1 Shrinkage

Shrinkage occurs when water evaporates from concrete. It is not a load-dependent deformation as creep is and that is why it is often called drying shrinkage. The evaporated water leads to a reduction of concrete volume which gives rise to tensile stresses when it is shrinking. If tensile stresses developed from the volume reduction are larger than the tensile stress capacity of the concrete cracks will develop. Drying shrinkage is active from a couple of weeks up to four years after casting and is governed by parameters such as concrete composition, water content, surrounding humidity, temperature and wind speed (Ansell et al 2012). Drying shrinkage is active a long time after hardening and not only in fresh concrete as with plastic shrinkage. The edge beam is a part that has more than 75 % surface exposed to...
surrounding environment, a condition that makes it vulnerable to drying shrinkage. There are ways to reduce the rise of shrinkage cracks and also measures to control the cracks. By introducing additives both plastic and drying shrinkage can be reduced and by using shrinkage reinforcement a fine crack distribution of small cracks can be obtained instead of a few coarse one.

### 3.2.2 Flexural (bending) cracks

During the service life of the bridge it will be subjected to loads that will give great moments at supports and near the mid span. The reinforcement in the concrete is supposed to carry the tensional stresses due to that it has not high tension capacity. At mid span it is the bottom part that is exposed to the highest stresses due to sagging and at supports those stresses are located at the top part due to hogging. Especially the edge beam can be vulnerable to tension stresses at the support. When a raised one is used the tension stress over supports is higher in the top of the edge beam than it is in the top of the bridge deck. It is a problem for the edge beam due to that it has a lower crack width limit than the bridge deck.

### 3.2.3 Carbonation

Two types of intrusion that lead to reinforcement corrosion are when carbonation and chloride penetrate the concrete (see figure 18). Carbonation intrusion is a natural procedure that occurs when carbon dioxide in the air reacts with calcium hydroxide in the edge beam concrete, giving calcite and water. In other words it leads to a backward reaction of the concrete that in the end gives calcite (lime) and is one of the basic substances. The calcite will be hard and it is not before the carbonation has reached the reinforcement and the level of pH in the concrete has decreased to below 9 that oxygen and moist can reach the reinforcement and thus leading to corrosion (Ansell et al 2012). Carbonation is therefore not harmful to non-reinforced concrete in the same extension.

### 3.2.4 Chloride intrusion

Chloride intrusion can lead to corrosion and is the one of these two considered being worst. This is a serious problem for bridges located in marine environments and where de-icing salt is used during the cold months of the year. Water splashes filled with chloride coming from vehicles hits the exposed edge beam. Even though it has not reached the reinforcement corrosion may well start if the amount of chloride reaches a certain value (see figure 18). This makes it hard for the designer to model the bridge against chloride attacks, it will also complicate the inspection of the edge beam due to that chloride intrusion is not as visible as carbonation intrusion (Ansell et al 2012).
3.2.5 Frost wedging

Two types of frost attacks can occur, one is sweet water and the other one is salt water attack. Existing moisture in the pore system, rain, sources of water as lakes and oceans and water splashes from vehicles will affect the exposed edge beam. The attack in itself is much the same for both types. When liquid water freezes to ice it expands in volume, leading to that cracks develop in the concrete as a result of the pressure. With the rise in number of cracks scaling of the concrete surface is initiated. There is a way of reducing cracks from frost attacks by introducing additives to the concrete during construction phase. By doing so pores with air are developed in the concrete. The pores allow water to expand without affecting the concrete. An air content of 5-6% is needed with a small distance between the pores so that the water has time to travel from pore to pore before it freezes to ice on the way, but it is important also to receive a compact concrete that do not allow water to saturate through the concrete too easy.

3.2.6 Deterioration in real life

The concrete cover of the reinforcement protects the reinforcement in more than one way. A suitable depth of cover is chosen due to high splitting stresses that arises from anchorage failure and splicing and in the same way a depth that can provide a protection against reinforcement corrosion. As soon as the concrete cover starts to deteriorate, the concrete fails to protect the reinforcement. The deterioration of concrete is a mixture of a lot of damaging actions. The ones mentioned above are the basic ones i.e. chloride- and carbonation intrusion, shrinkage, thermal contraction, flexural cracks and frost attacks.

Chloride and carbonation intrusion make it easier for water to penetrate. In some cases cracks develop in the edge beam before the bridge has been put in use, this is the case when plastic shrinkage and thermal contraction give rise to cracks during the construction phase.
water ingress into the concrete it will freeze and thaw during the colder months of the year, leading to micro cracks that will expose the reinforcement in due time if permitted.

When the reinforcement is exposed it is a matter of environmental issues that decides how fast the reinforcement will corrode. Corrosion of reinforcement leads to a volume expansion due to that rust has a greater volume than steel. More cracks are developed leading to that more reinforcement is exposed and due to the volume expansion of the reinforcement the anchorage is reduced considerably which in the end affect the bearing capacity of the structure.

### 3.3 Preventive maintenance

There are two types of measures to reduce the influence of deterioration processes. Preventive maintenance work before the edge beam has been set into work and corrective maintenance when the damage is already presence and measures are needed to neutralise the edge beam. Common types of preventive measures are impregnation, cathodic protection of reinforcement and stainless steel reinforcement.

#### 3.3.1 Impregnation

The edge beam is impregnated by a fluid that reduces penetration of concrete pores (Weber 2014). By impregnating, measures are done to prevent the intrusion of water and chloride which in turn reduce the risk of freeze- and thaw attacks as well as corrosion induced by chloride intrusion.

#### 3.3.2 Cathodic protection of reinforcement

By introducing either a grid anode or a rod anode the corrosion of reinforcement that is an electrochemical process can be inhibited. This is done by reducing the electrochemical potential which in turn prevents the reinforcement to corrode (Maglica 2012).

#### 3.3.3 Stainless steel reinforcement

The edge beam is located in an exposed environment and one solution could therefore be the use of stainless steel reinforcement. However this is a solution that would increase the investment cost significantly and is probably why it has not been used in a great scale before (BSSA 2013).
3.4 Corrective maintenance

3.4.1 Concrete repair

As a consequence of erosion that exposes the reinforcement the concrete can be repaired in two ways, either by using shotcrete or by casting it manually. Before new concrete can be applied to old concrete all damaged concrete and rusted reinforcement is removed with the help of a demolition hammer. By giving it a rough surface the new concrete is added so that good adhesion is obtained (Sobhit 2013).

3.4.2 Crack injection

By injecting cracks with resin made of polymers the intrusion of harmful substances to the reinforcement can be reduced. To begin with the surface of the crack is brushed clean from loose material with dry air so that the polymers attach good. Thereafter the resin is pumped into the crack to obtain a good isolation. This process is only done when the crack has finished moving, in those cases when it is still active an injection of resin will only give rise to a new crack (Concrete society 2014).

3.4.3 Replacement

In some cases the edge beam is damaged to a point that there is no benefit in just repairing the edge beam, in those cases a replacement is done instead. This is done by removing the whole edge beam and replacing it with a new one. The choice on whether to do a repair or replacement work is done by the person(s) inspecting the bridge.

A replacement work is more comprehensive than a repair work and is often affecting the surrounding traffic flow. It is quite common that lanes going cross the bridge are closed down or that allowed traffic speed is reduced. Nowadays new solutions have been developed. In figure 19 it can be seen how a solution can make it possible for the building workers to work from the outside of the bridge. A scaffolding system takes less space on the bridge deck making it possible to maintain a nicer traffic flow than with previous replacement methods (MoldTech 2014).

![Figure 19](image_url)  Scaffolding system for replacement of edge beams (MoldTech 2014)
4 Building site visits

4.1 Case study 1

In Askesund just outside of Örebro Skanska are building a short span bridge with a length of 13m on road 50 over road 49 by Stjänsund. It is an ordinary frame bridge with an integrated edge beam that will make it possible to drive on and off road 49 and 50 (see figure 20).

4.1.1 Edge beam

The edge beam will be poured at ground then lifted up to the bridge deck when it has hardened. At bridge deck level the two parts will be cast together (see figure 21). Making it a prefabricated edge beam element that works as an integrated part after they have been cast together. It is comfortable to work with due to that it is a pre-fabricated edge beam until a certain point of time. It is easier to vibrate the concrete on ground and nothing sipper out from the formwork of the edge beam to the bridge deck. Working on ground means that less man power is needed and it is more comfortable to work on ground than it is to work on bridge deck level. A result of this is better quality of the concrete for the edge beam elements and better working circumstances, two advantages constructing it in this way when the span of the bridge is short and the elements are constructed in one piece.
The attitude towards the pre-fabricated edge beam has mostly been positive because of the easy replacement process. The type used in Askersund will not work as a pre-fabricated edge beam in that sentence but the high quality of concrete may aggravate the intrusion of harmful substances. It may be in need of less maintenance work than an ordinary integrated edge beam.

4.2 Case study 2

In Rotebro north of Stockholm NCC are building two new bridges on European road 4 (E4) with a length of 325m and three lanes on each bridge (see figure 22). The pressure on the existing bridges were increasing and wearing lead to that it was needed to build new bridges to manage these pressures. A new feature will be the new H4 railing, to manage the increasing traffic pressure. These are a big part of the infrastructure and connect Stockholm to Uppsala, a reason to why the influence on traffic both on the bridge and under where two existing roads and railway tracks are located should be kept to a minimum. The daily traffic that travels is approximately:

- 70,000 vehicles on the bridges.
- 25,000 vehicles beneath the bridge.
- 600 trains beneath the bridge.

The solution to maintaining an acceptable traffic flow is to build temporary supports for the new bridge so that two useable bridges are presence during most of the building process. When it is built the temporary supports are removed and the it is slowly shifted into place, during this period of three weeks the traffic in both directions will be directed to one bridge.
This will be done sometime during the summer when the traffic flow is lower than usual (Trafikverket 2014).

Figure 22  Two NCC bridges connecting Stockholm with Uppsala (Trafikverket 2014)

4.2.1 Edge beam

The edge beams in Rotebro are larger than regularly used types and with a larger amount of reinforcement, in this case 13ϕ16mm which is approximately 1% of the cross sectional area (see figure 23). It was poured afterwards and integrated to the already hardened bridge deck. It was thought that the decision of using a more robust railing would affect the design of the edge beam according to TRVR Bro 2011 B.1.12.2.1 but this was not the case since prescribed minimum amount of reinforcement in TRVK(R) Bro 2011 was not exceeded. The choice of using 13ϕ16mm is based on working circumstances, the size of the edge beam and railing bolts (Pettersson NCC 2014). The goal was to obtain a fine crack distribution, which will be obtained for a larger amount of reinforcement (Johansson 2009). An inclination of the bottom side is a new feature for these bridges, it could be an indication on that the drop nose do not work as well as it should.
4.3 Case study 3

In Kallhäll, located in the outer parts of Stockholm, Skanska is for the time being building the new Landskapsbron. It is a wildlife bridge that connects two nature reserves on each side of the railway track. It is one of many bridges along Mälarbanan where two new railway tracks are planned to be built, an increase from the two existing ones to four railway tracks (see figure 24-25). The goal is to separate the commuter trains from rest of the train traffic (Trafikverket 2014).
As mentioned before it is a wildlife bridge which means that the difference between Landskapsbron and other road bridges is that it will be filled with earth to a height of approximately 0.8m and have a width of 50m.

![Figure 25 A wildlife bridge crossing Mälarbanan (Trafikverket 2014)](image)

### 4.3.1 Edge beam

The edge beam used has a height of 1.1m to withstand the earth filling and an inclination of 10° at two locations on the outside. It was first planned to look like a straight wall but an inclination of the side was chosen instead by the involved architect and owner of the bridge (Broman 2014). Test on edge beams with different inclinations has been performed in previous work and showed that with increasing inclination the amount of air pores increases as well (Andersson et al 2009). This makes it easier for substances to penetrate the covering surface of the concrete. The cross section of the edge beam can be seen in figure 26.
Cracks would develop in the edge beam over supports due to the high dead load. It would also complicate the pouring of the bridge deck if they were cast together. To simplify the working process it was decided that the bridge deck was poured first then the edge beam was cast and integrated together. Otherwise a scaffolding system that reaches over the edge beam would be needed, which in turn would complicate vibration of the concrete. When new concrete is cast together with old concrete, large temperature differences develop which may lead to thermal contraction cracks. To solve this problem, joints were introduced in the edge beam all along the bridge (Nagy 2014). The purpose of the joints is to narrow the cracks to the locations of the joint (see figure 27). This method has proved to have a drawback, they will be narrowed to the joints but this will in turn lead to the development of a few coarse cracks going into the bridge deck (Sandelius 2014), in that way eliminating the purpose of the joints. A good distribution of smaller cracks is preferred rather than to have a few coarse ones according to TRVK Bro 2011 D.1.4.1.6.
Figure 27  Elongation of the edge beam with installed joints over support and along the bridge span
5 Design study

When designing an edge beam in Sweden the rules and regulations in Eurocode and TRVK(R) Bro 2011 are used. Some rules from TRVK(R) Bro 2011 are listed below to ease the understanding of how it should be designed. K and R in TRV- stands for demands (krav) and advices (råd). Following part is an interpretation of different chapters and paragraphs of the codes and regulations.

When it comes to geometry of the edge beam, the regulations can be found in TRVR Bro 2011 D.1.2.7.3. It states that the height and width of edge beam shall be chosen so that a sufficient load distribution in the bridge deck is obtained when point loads are applied to the bridge deck, but the height and width should not be less than 400mm. The edge beam shall be designed to withstand an impact to the railing.

In TRVK Bro 2011 D.1.4.1.6 it is stated that the edge beam has to be dimensioned with an amount of reinforcement that gives a good distribution of cracks. The edge beam is located in road environment leading to that the crack width should be limited to 0.15mm.

In TRVK Bro 2011 D.1.3.1 it stands that concrete structures should be given a design that makes it possible for water to run off horizontal surfaces, but not onto visible vertical surfaces.

More codes and regulations regarding the inclination, reinforcement amount etc. exists but the ones stated in this chapter are thought to be the most fundamental for designing the edge beam. It should be highlighted that following one rule or regulation should not confine another rule or regulation e.g. if minimum amount of reinforcement is used, a crack width control should also be in place. Another thing is that if it is not stated in TRVK(R) Bro 2011 it does not necessarily mean that it is forbidden (Ronnebrant 2014).

5.1 Controls for an integrated edge beam

The design study example was performed for a typical integrated concrete edge beam. The purpose of this was to see how well the most popular type in Sweden is adapted to the codes and regulations. This was carried out by applying the controls that are done when dimensioning the edge beam.

Different types of failure possibilities can be applied when designing the edge beam for a local control of an impact load. Yield failure of the bolt and railing together with anchoring failure of the bolt are dimensional factors. As pointed out before it is designed to support the
railing during an impact. In SS-EN 1991.2 section 4.7.3.3.(2) it is said that the part supporting the vehicle restraint system (railing) should be designed for at least 1.25 times the characteristic load capacity of the vehicle restraint system, this is the safety precaution value (α). It can be found in each country’s national annex. In TRVFS 2011:12, Ch. 6 §11 it is stated that a value of 2.0 should be chosen for bridges in Sweden.

The load during an impact is distributed between two railings posts. The height/eccentricity of the load is seated at approximately the same level of the guard rail (see figure 28). To obtain the dimensioning impact force (\(F_H\)) for the edge beam the lowest load capacity of anchoring failure of the bolt group as well as yielding of bolt and railing were chosen.

![Figure 28](image)

**Figure 28**  
A prototype of the edge beam used for impact control of the vehicle restraint system

### 5.1.1 Failure of threaded bolt

Failure in the threaded bolt takes place when the bolt group starts to yield. The characteristic tension force capacity of bolts (\(F_v\)), the number of bolts (n) and internal level arm (a) of the bolt group are the basic parameters that govern the capacity of the bolts. The moment capacity (\(M_{bolt}\)) of the bolt group can be calculated and in turn be set equal to the moment acting on the bolt group.
With the help of the bolt group moment resistance and the moment acting on the bolt group the design impact load for failure in the threaded bolt \( F_{H.bolt} \) can be calculated [see appendix E.3].

\[
F_{H.bolt} = \alpha * \frac{M_{bolt}}{e_{bolt}} \tag{5-2}
\]

Where \( e_{bolt} \) is the eccentricity of the horizontal impact load to the top of the edge beam. In SS-EN 1991.2 section 4.7.3.3.(1) it is stated that the eccentricity should be located 100mm below the top of the railing or 1.0m above the carriage way. Even though these values are recommended by the Eurocode it is common that the distance between the top of the edge beam up to the guard rail is chosen instead to be on the safe side.

### 5.1.2 Anchoring failure of bolt

A possible scenario during an impact to the railing is that the bolts are pulled out of the edge beam due to the great impact load that the vehicle produces. Therefore the capacity of the anchoring force of the bolt group has to be checked. By applying the section about ultimate bond stress (8.4.2) and anchorage length (8.4.3) in SS-EN 1992.1.1, the design horizontal force acting on the edge beam can be calculated.

By using the design tensile strength of the concrete \( f_{ctd} \) and coefficients for adhesion capacity of rebar \( \eta_1 \) and rebar size \( \eta_2 \), the ultimate bond strength \( f_{bd} \) is calculated.

\[
f_{bd} = 2.25 * \eta_1 * \eta_2 * f_{ctd} \tag{5-3}
\]

With the help of the ultimate bond stress, the required anchorage length \( l_{b.rqrd} \) and bolt diameter \( \phi \) equation 8.3 in ss-en 1992.1.1 can be readjusted to get the design stress \( \sigma_{sd} \) of the bolt.

\[
\sigma_{sd} = \frac{4 * l_{b.rqrd} * f_{bd}}{\phi} \tag{5-4}
\]

The design stress for the bolt is used to calculate the characteristic anchorage load capacity of the bolt group. By multiplying the nominal stress area of the bolt with the design stress the
characteristic load is calculated. The load is applied to the same equations as in previous section to calculate the design force of the edge beam [see appendix E.3].

### 5.1.3 Failure in the railing

The last part of the vehicle restraint system that is checked is failure in the railing itself. First the moment resistance of the railing ($M_r$) is calculated with the help of the plastic bending moment ($l_{side}^3/4$, where $l_{side}$ stands for the side length of the railing) and the design yield limit ($f_{ud}$) of the railing.

$$M_r = \frac{l_{side}^3}{4} \times f_{ud} \tag{5-5}$$

Then the moment from the horizontal load of the impact ($M_s$) is calculated by multiplying the horizontal force ($F_{HR}$) with the eccentricity from load to footplate of railing ($e_{HR}$).

$$M_s = F_{HR} \times e_{HR} \tag{5-6}$$

If these two are set equal to each other the design load acting on the edge beam can be calculated [see appendix E.3].

$$M_s = M_r \rightarrow F_{HR} = \frac{M_r}{e_{HR}} \tag{5-7}$$

### 5.1.4 Bending in the edge beam

The lowest load capacity ($F_H$) from chapter 5.1.1-5.1.3 will be the dimensioning one that gives rise to bending and torsion in edge beam and bridge deck. The dimensioning moment ($m_{ed}$) and normal force ($n_{ed}$) from the impact load is calculated first. The eccentricity ($e_m$) is the length from the load to the middle of the chosen section and $c_c$ is the distance between two railing posts.

$$m_{ed} = \frac{F_H \times e_m}{c_c} \tag{5-8}$$

$$n_{ed} = \frac{F_H}{c_c} \tag{5-9}$$
The design moment and normal force are used to get the required amount of bending reinforcement \( (A_s) \). The transversal amount of reinforcement should be calculated for a section in the edge beam and one in the bridge deck if the effective heights \( (d) \) differ (see figure 29) [see appendix E.3].

\[
A_s = \frac{m_{ed} + \frac{n_{ed} \times d}{2}}{f_{s,yd}}
\]

\( m_{ed} \) is the design moment and \( n_{ed} \) is the design normal force. The subscripts \( ed \) indicate edge beam and \( f_{s,yd} \) is the design yield limit of the reinforcement.

5.1.4.1 Degree of capacity utilization

The degree of capacity utilization \( (\eta) \) is possible to calculate when the required amount of reinforcement has been decided. \( \eta \) is calculated by dividing the required amount with the used amount of reinforcement \( (A_{s,used}) \).

\[
\eta = \frac{A_s}{A_{s,used}}
\]

The capacity of utilization is used to adjust the anchorage length of the reinforcement. It can be calculated with the equation used in previous chapter, (6.1.2) failure in the bolt. The design stress is calculated with the help of the design yield limit of the reinforcement \( (f_{yd}) \).

\[
\sigma_{sd} = f_{yd} \times \eta
\]

5.1.5 Shear and torsional resistance

The horizontal impact load will give rise to a shear force and a torsional force and moment in the edge beam (see figure 30). The shear force \( (V_{Ed,F}) \) and the torsional moment \( (T_{ed}) \) are calculated according to eq. 6.26 and 6.27 in SS-EN 92.1.1 [see appendix E.3].
The longitudinal amount of reinforcement needed in the edge beam due to torsion can be calculated with eq. 6.28 in SS-EN 92.1.1. Where $\theta$ stands for the crack inclination, $u_k$ for the perimeter of the reinforcement, $A_k$ for the area within the frame of the reinforcement and $f_{yw,d}$ for the design strength of steel.

$$A_{sl} = \frac{T_{ed} \cdot cot\theta \cdot u_k}{2 \cdot A_k \cdot f_{yw,d}}$$

(5-13)

The transversal amount of reinforcement can be calculated by applying the shear force and the torsional force in combination with the effective height ($d$) to eq. 6.8 in SS-EN 92.1.1.

$$A_{s,v,Ed} = \frac{V_{Ed,T} + V_{Ed,F}}{0.9 \cdot d \cdot f_{yw,d} \cdot cot\theta}$$

(5-14)
5.2 Cracks over supports

Cracks over supports is a well-known problem for edge beams, this was the case for the railway bridge over Ångermanälven, mentioned in the Abstract. It was of interest to know if it was possible to keep the crack width \( w_k \) at an acceptable level of 0.15mm and in that case how much reinforcement will be needed in the edge beam, over supports for a regular road bridge. By applying eq. 7.8 in SS-EN 1992.1 (7.3.4), the crack width was calculated with the stress \( \sigma_{s,eb} \) acting in the center of gravity of the reinforcement in the edge beam.

The stress was interpolated from the bridge deck of a real life bridge located in Kista, Stockholm (see figure 31). By using the same excel sheet that was used when dimensioning the amount of reinforcement in the top part of the bridge deck of a real life bridge, the stress in the edge beam could be interpolated for a specific amount of reinforcement in the bridge decks top part [see appendix E.4-5].

![Figure 31 Cross section of bridge deck over support for Kista bridge](image)

5.2.1 Crack width for 7- and 9ϕ16mm rebars

The crack width for an edge beam with 7ϕ16mm and 9ϕ16mm was calculated to see how large crack width is obtained when using the advised minimum amount of reinforcement according to TRVR Bro 2011 D.1.4.1.6. The results are listed in table 2. None of the two cases has a crack width under 0.15mm.

<table>
<thead>
<tr>
<th></th>
<th>7ϕ16mm</th>
<th>9ϕ16mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w_k ) [mm]</td>
<td>0.272</td>
<td>0.218</td>
</tr>
</tbody>
</table>

Table 2 Calculated crack width according to equation 7.8 in SS-EN 92.1.1 for 7ϕ16mm and 9ϕ16mm
By increasing the amount of reinforcement in the top part of the bridge deck over support the received stress in the bridge deck can be reduced, leading to that the stress in the edge beam also will be decreased (see figure 32). The goal was to investigate if the crack width limit could be reached by increasing the amount of reinforcement in the bridge deck. In table 3 it can be seen that the crack width of 0.15mm can be reached both for 7ϕ16mm and 9ϕ16mm. An increase of 40% i.e. from 843cm$^2$ to 1180cm$^2$ gave a crack width of 0.156mm when using 7 rebars. When increasing with 20% i.e. from 843cm$^2$ to 1012cm$^2$ a crack width of 0.154mm was obtained when there are 9 rebars in the edge beam.

Table 3  Crack widths when the amount of reinforcement in the top part of the bridge deck over support is gradually increased

<table>
<thead>
<tr>
<th>Increase of reinforcement</th>
<th>A.s in bridge deck [cm$^2$]</th>
<th>7ϕ16mm</th>
<th>9ϕ16mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>10%</td>
<td>927</td>
<td>0.235</td>
<td>0.183</td>
</tr>
<tr>
<td>20%</td>
<td>1012</td>
<td>0.205</td>
<td><strong>0.154</strong></td>
</tr>
<tr>
<td>30%</td>
<td>1096</td>
<td>0.178</td>
<td>0.128</td>
</tr>
<tr>
<td>40%</td>
<td>1180</td>
<td><strong>0.156</strong></td>
<td>0.107</td>
</tr>
<tr>
<td>50%</td>
<td>1264</td>
<td>0.136</td>
<td>0.088</td>
</tr>
</tbody>
</table>

5.2.2 Maximum amount of reinforcement in the edge beam

A check was performed to see if it is possible to reduce the crack width in the edge beam to an acceptable level by theoretically adding as much reinforcement as possible in the edge beam over a support (see figure 32). The crack width was calculated with maximum amount of rebars for sizes of 16mm, 20mm, 25mm and 32mm. The calculated crack widths are listed in table 4 for the four cases. Only for the first case with 24ϕ16mm was a crack width of acceptable level obtained. This indicates that even though the amount of reinforcement (area) is equally large it is preferable to choose a rebar with smaller diameter.

Table 4  Calculated crack width for rebar sizes of 16-, 20-, 25- and 32mm

<table>
<thead>
<tr>
<th></th>
<th>Φ16mm</th>
<th>Φ20mm</th>
<th>Φ25mm</th>
<th>Φ32mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebars per level</td>
<td>4</td>
<td>4</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Levels</td>
<td>6</td>
<td>5</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>w.k [mm]</td>
<td>0.149</td>
<td>0.244</td>
<td>0.273</td>
<td>0.328</td>
</tr>
<tr>
<td>Reinforcement area (mm$^2$)</td>
<td>4825</td>
<td>6283</td>
<td>5890</td>
<td>4825</td>
</tr>
</tbody>
</table>
5.3 Anchorage of an pre-fabricated edge beam

The pre-fabricated edge beam should be designed to be able to withstand an impact to the railing (see figure 33). To investigate this more in detail, the same design impact load ($F_H$) for the edge beam example in chapter 6.1 was used i.e. a H2 railing. The impact load was tested for three different heights of the bridge deck ($H_{bd}$) to determine what height is needed for the bridge deck to withstand an impact. There are two criteria’s that needs to be fulfilled, the first one is that an impact gives rise to a compression zone ($h_{cc}$) in the bottom side of the edge beam i.e. the compression zone has to fit with concern to the effective height from bottom to anchorage bolt. The other criteria is that the two M24 bolts around each railing post that was used, with a distance of 500mm between them should be able to withstand the tensional forces acting on the anchorage during an impact. The compression part and resistance of the anchoring bolts are therefore the parameters that govern the height of the bridge deck [see appendix E.6]. A possible increase of height gives higher costs for the bridge and could be a reason to consider it as a poor solution.
For different heights of the bridge deck the compression part acting in the edge beam was calculated. Starting with a random value for the height of the compression part, the force acting on the anchorage connection was calculated. By doing iterations the goal was to reach convergence for the height of the compression. For the calculated height of the compression zone the force from impact acting on the anchorage was calculated. Thereafter it was possible to say if the compression zone was too big and if the load capacity of the anchorages could withstand an impact to the railing. For the chosen bridge deck height the stress obtained in the bolts were calculated to determine if the structure was over reinforced, which it was not in this case [see appendix E.2].

For a height of 0.30m of the bridge deck convergence was reached after five iterations, giving a height of 29mm to the compression part and also fulfilling the load capacity of the anchorage [see table 5]. A solution would have been to strengthen the anchorage but that would increase the risk of obtaining an over reinforced structure.

<table>
<thead>
<tr>
<th>Height of bridge deck [m]</th>
<th>Hbd</th>
<th>0.25</th>
<th>0.27</th>
<th>0.30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of iterations</td>
<td>n</td>
<td>6</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Height of compression part [mm]</td>
<td>hcc</td>
<td>34</td>
<td>30</td>
<td>29</td>
</tr>
<tr>
<td>resistance &gt; load</td>
<td></td>
<td>FALSE</td>
<td>FALSE</td>
<td>TRUE</td>
</tr>
</tbody>
</table>


5.4 Distribution of point load

When an integrated edge beam is used for a bridge it will contribute to the stiffness of the cantilever and help distributing concentrated loads ($P$). By applying eq. 2-14 in Sundquist 2011 the clamping moment ($m_{i,\text{max}}$) in the section above the support beam can be calculated for a concentrated load on the cantilever plate[see appendix E.8]. Where $k_t$ stands for the coefficient taking the cantilever thickness into account, $a$ stands for the lever arm between the point load and the support, $t$ is the thickness of the cantilever and $I$ is the moment of inertia of the edge beam.

$$m_{i,\text{max}} = \frac{t}{4} \sqrt{\frac{k_t \cdot a}{tI}}$$

(5-15)

For this study two cases were performed, the first case was when an integrated edge beam was used and the second case was when a not existing edge beam type was used (see figure 34). In the second case Pucher diagrams of influence surfaces were used to calculate the moment at the support coming from traffic [see appendix E.7].

![Figure 34: The cantilever with the edge beam in the left picture and the cantilever without a real edge beam in the right picture](image)

The reason for this is that the equation provided in Sundquist 2011 cannot be used when the moment of inertia of the edge beam is zero. A real life bridge on European road 6 (E6) close to Dynekilen [see appendix D.2-5] was used for this part of the design study (see figure 35). By applying these 2 cases to a real life bridge a comparison of how good the edge beam distributes concentrated loads and if exclusion of the edge beam could be an alternative choice instead.
Greater moments in the cantilever were developed when the edge beam was included. Approximately 25% more reinforcement was needed for the cantilever with an edge beam (see table 7). This indicates that less moment at the support is needed when the edge beam is not used. An explanation of this is that when the Traffic Load Model 1 (LM1) (see figure 36 and table 6) is applied to the cantilever the edge beam is not included in the Pucher control i.e. it do not contribute to the total moment with its dead load. Even though the edge beam helps giving a better distribution of the moment coming from traffic it also contributes with its dead weight. Which for this bridge results in higher clamping moment for when an edge beam is used.

Figure 35  Cross section of the steel concrete composite in Dynekilen

Figure 36  Traffic load model 1 (Sundquist 2009)
### Table 6  Loads for LM1 according to Eurocode

<table>
<thead>
<tr>
<th>Lane</th>
<th>A [kN]</th>
<th>p [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane 1</td>
<td>300</td>
<td>27</td>
</tr>
<tr>
<td>Lane 2</td>
<td>200</td>
<td>7,5</td>
</tr>
<tr>
<td>Lane 3</td>
<td>100</td>
<td>7,5</td>
</tr>
</tbody>
</table>

### Table 7  Moment and amount of reinforcement for Ultimate Limit State (ULS) and Serviceability Limit State (SLS) for Dynekilen bridge with and without an edge beam

<table>
<thead>
<tr>
<th></th>
<th>With edge beam</th>
<th>Without edge beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic moment [kNm/m]</td>
<td>$M_{LM1}$</td>
<td>85,52</td>
</tr>
<tr>
<td>Total moment – uls [kNm/m]</td>
<td>$M_{uls}$</td>
<td>211,8</td>
</tr>
<tr>
<td>Total moment – sls [kNm/m]</td>
<td>$M_{sls}$</td>
<td>36,86</td>
</tr>
<tr>
<td>Amount of reinforcement – uls [cm²]</td>
<td>$A_{s,uls}$</td>
<td>25</td>
</tr>
<tr>
<td>Amount of reinforcement – sls [cm²]</td>
<td>$A_{s,sls}$</td>
<td>10,2</td>
</tr>
</tbody>
</table>
6 Discussion

6.1 Codes and standards

The Swedish nation annexes are quite extensive in comparison to other countries. Denmark for example is one of the countries who have a nation annex that is not as extensive as the Swedish one, in one way leaving more degrees of freedom for the engineer. This is thought to let the engineer’s creativity flow and leave more space for improvement.

6.2 Bridge cases

6.2.1 Implementation of edge beam elements

If the integrated edge beam is proved to be the best solution for short span bridges, then prefabricated elements is thought to be a suitable solution. This is due to that it was possible to cast the edge beam in one piece but it is left to see how well this solution would work for a long span bridge where the edge beam has to be cast in pieces. The vertical joint between two edge beam elements is a problematic area due to that the reinforcement is double spliced (see figure 37). The element length are governed by the lifting capacity meaning that it is not possible to lift too long elements without risking that they will deform during the process. So this method will lead to the development of good quality concrete when it comes to the elements and bad quality when it comes to the joints. In that way destroying the purpose of introducing pre-fabrication when it comes to longer bridges. If it is not possible to come up with a good solution for the joints, it is fair to say that this type of edge beam will only be applicable for short span bridges.
6.2.2 H4-railing

The H4-railing was introduced for the two bridges in Rotebro, more robust than the regularly used H2-railing. In the first part of the design study it was clear that the resistance of the vehicle restraint system is dimensional for the edge beam. This was also the case when edge beam calculations from companies were observed. The resistance of the railing was in all cases lower than the capacity of the bolts. The H4-railing has a higher resistance capacity leading to that the dimensioning impact load is greater than for a H2-railing. Even though the choice of railing did not affect the design of the edge beam. Which indicated that even though it is said that the vehicle restraint system is dimensional for the edge beam this is not the case, minimum heights, widths and amount of reinforcement is governed by the codes and regulations instead. What can be said though is that with time newer and more robust railings will be developed and then it could be the case that it will affect the design of the edge beam.

6.2.3 Landskapsbron

The big edge beam used for Landskapsbron was problematic to cast for the workers and it also gave rise to cracks due to high dead loads and temperature differences as a result of the complicated casting process. The formwork and scaffolding system for larger edge beams will be more complicated to manufacture which will in turn affect the vibration of the edge beam.

It was the architect’s choice to have an inclination of the outside part. This made the formwork harder to construct compared to the first design when the edge beam would have the design of a plain wall. However it is a small price to pay if it fulfills its aesthetical purpose during the life time of the bridge. How well the edge beam and bridge will withstand deterioration processes and if the design of the edge beam in any way will contribute to this remain for the future to see. What can be said now is that it has been proven that with increasing inclination of the edge beam side there will be an increase of pores in the surface which will facilitate the penetration of concrete. One assumption is that it will not fulfill its aesthetical purpose due to deterioration of the outside part. This in combination with dirt coming from the fill will give another aesthetical view than was first expected.

It is possible to discuss whether it was a good idea or not to introduce joints in the edge beam but from a general point of view it would have maybe been better to use stainless steel
reinforcement. Due to the size of the edge beam it is thought that it will crack anyway and then the best preventive maintenance would be the use of stainless steel reinforcement, in that way reducing the need of corrective maintenance that affects underlying train traffic.

6.3 Cracks over support

Real life stresses from a bridge in Kista, Stockholm were used for the calculation of crack widths in the edge beam over supports. For this type of bridge the design study gave a good approximation of how much reinforcement was needed not to exceed the crack width limit for an edge beam. The first result in this study showed that when a minimum amount of reinforcement and when a common type of solution was used (7ϕ16mm and 9ϕ16mm respectively) a crack width of 0.272mm and 0.218mm was obtained which is not acceptable. It shows that a solution of 7-9ϕ16mm would not work for the whole length of the bridge i.e. at least not over the supports.

It is possible to increase the amount of reinforcement locally over the support to decrease the obtained crack width in the edge beam but this would aggravate the casting process. Assumed that there is no possibility to keep the crack width at an acceptable level for the integrated edge beam, a possible solution could be the use of stainless steel reinforcement locally over supports. If not this could be a reason to use another type of edge beam assumed that it fulfills the crack width limitations better.

6.4 Anchorage of pre-fabricated edge beam

The design impact load was obtained from the anchorage part of the design study, for which a regular H2 railing was used. One solution that excluded the need of a thicker bridge deck was if the anchoring is strengthened but by doing so the risk of obtaining an over reinforced structure is high.

With increasing traffic flow and larger vehicle transport weights more robust railings will probably be used. Increasing capacity of the railing gives a higher design impact load for the dimensioning of bridge deck height. This would increase the height more than when a H2 railing is used, as was the case in this part of the design study. Overall an increase of the bridge deck would increase the cost for the whole bridge and decrease the probability of choosing the pre-fabricated edge beam.

6.5 Clamping moment in a cantilever

Two different methods were used for the two cases and it is possible to discuss the reliability of doing so but in this case they were thought not to be farfetched from each other.

With an edge beam the moment at the section over the support coming from traffic was lower than when the not existing edge beam type was used. The total moment coming from both traffic load and dead load was higher for the bridge deck with an edge beam than without.
Even though the influence from the edge beam is visible and it is clear to see that it distributes the traffic load better along the section above the support it seems that the dead load plays a significant role for the total moment. This shows that it could be a good idea to exclude the edge beam, a lower clamping moment is obtained and there is no extra cost of casting an edge beam. How large influence the dead load from the edge beam has on the moment when the cantilever is larger and more traffic load can be contained has not been included in this design study.
7 Conclusions

7.1 Codes and standards

The fact that a certain specification or rule is not stated in the codes or standards does not necessarily mean that it is forbidden. As long as the ones that are indicated there are fulfilled it is the engineer's task to handle adequately the existing degrees of freedom in order to make the correct decisions for the edge beam.

When designing the edge beam a rule or regulation should not interfere with another rule or regulations e.g. it should not be approved to dimension an edge beam with only minimum amount of reinforcement, it should also be designed to obtain a good crack distribution but as the crack width investigation showed this is harder done than said.

7.2 Bridge cases

7.2.1 Askersund

Due to that the edge beam is poured on ground level the vibration process will be facilitated and there is no risk of concrete pouring out to the bridge deck which happens when the edge beam is cast together with the bridge deck. The quality of the concrete is better but approximately to the same price. Maybe less man hour but in the same way a ground platform for pouring of the edge beam will be needed as well as a crane to lift the pre-fabricated edge beam.

7.2.2 Rotebro E4

It was clear that the more robust H4 railing gave a larger dimensioning impact load for the edge beam. However the increased impact load in this case did not significantly influence the amount of reinforcement which instead was governed by other factors.
It was proved by Johansson and Lantz 2009 that the edge beam length does not affect the crack width and distribution to a remarkable extent. However increased amount of reinforcement has proven to give a finer distribution of cracks. A good distribution is preferred rather than obtaining a few large cracks.

### 7.2.3 Landskapsbron

Landskapsbron is a real life example on that large edge beams complicates the working process which indirectly affects the quality of the edge beam. In this case the large edge beam made it impossible to cast the edge beam and bridge deck at the same time which was why joints were introduced, that and also the high dead weight. Joints that reduced the cracks to one location but in the same time considered to give larger crack widths than when a good distribution of cracks is obtained all along the span of the bridge.

### 7.3 Design study

#### 7.3.1 Cracks over support

Even if the edge beam is theoretically packed with as much rebars possible with concern to concrete covering and rebar spacing in vertical and horizontal direction the crack width limit will still be exceeded for the thicker rebars. Only for 24\(\phi16\text{mm}\) could the crack width be limited to beneath 0.15mm. Indicating that the formula used in SS-EN 92.1.1 (7.4.3) gives a larger crack width for thicker rebars, even if the total amount of reinforcement \([\text{mm}^2]\) is equal or higher.

An increase of reinforcement in the bridge deck decreases the stress that acts in the edge beam. The test showed that it is possible to reduce the crack width for the edge beam to an acceptable level if the amount of reinforcement in the bridge deck is increased significantly.

Both of these solutions are theoretical solutions that would not have worked in real life situations. It would be impossible to vibrate the concrete with that many rebars located in the bridge deck and edge beam over supports.

#### 7.3.2 Anchorage of pre-fabricated edge beam

The height of the bridge deck is governed by the height of the compression zone acting in the bridge deck during an impact. The two anchorage bolts that were used showed that a height of 0.30m for the bridge deck was needed. The compression zone that emerges after an impact did not fit for bridge decks with smaller heights. More bolts could be used but that would not decrease the height of the compression zone and could in turn give an over reinforced structure.
7.3.3 Clamping moment in a cantilever

For the bridge in Dynekilen it is clear that if the false edge beam type would have been chosen instead of the integrated edge beam the moment over the support would have been reduced. The edge beam gives a better distribution of the traffic load but the dead load of the edge beam in turn increases the moment significantly for this bridge.

7.4 Further research

- The pre-fabricated edge beam elements used in the Askersund project could be a new research topic. Provided that it results in better working circumstances and better concrete quality, the question is how this could be applied for bridges with longer spans. In Askersund the edge beam was poured in one piece but for longer spans it would not be possible to cast the edge beam in one piece and lift it up. The length of each edge beam element is governed by the lifting capacity i.e. how large elements that can be lifted without deforming during the procedure. The concrete quality of the joints between two edge beam elements will be lower than for the pre-fabricated elements. So the question comes down to how the solution for the joints would look like.

- To prove that the false edge beam is a better choice than the integrated concrete edge beam more checks than the one done in this report should be performed. In this report it was proved that for one control the required amount of reinforcement was higher for the bridge in Dynekilen if an edge beam is used. It may well be the case that the height of the bridge deck and amount of reinforcement will be higher for when an edge beam is not used. For further research a finite element programme could be used so that the same method is applied for the two cases.

- The steel edge beam was not investigated in this master thesis. If investigations are planned for the near future it would be of interest to know if it is used anywhere in the world and how well it works. Price estimation should be done to know if it even is price optimal, including price for material, cost for constructing it and repair and replacement cost. Other aspects will be if it fulfills anchorage criteria’s and how well the joint between the bridge deck and edge beam resist intrusion.

- When a pre-fabricated edge beam is used it will not contribute to the distribution of concentrated loads and even if it does in practice it will hardly be the same as when an integrated edge beam is used. With reduced distribution of concentrated point loads and a dead load from the edge beam that participates in the bridge structure it is of interest to know if the pre-fabricated edge beam will increase the thickness of the bridge deck and amount of reinforcement.

- With increasing transport weights and faster vehicles the railing capacity will be in need of an increase. It will be of interest to know how improved capacities of railings may affect the outcome of the edge beam design and especially the pre-fabricated edge beam.

- In this master thesis solutions as stainless steel reinforcement and durable concrete were not evaluated as a solution to frequent problems i.e. how good it withstands deterioration processes and how large LCC it has.
Another solution that was not evaluated in this report is the use of pre-tensioned edge beams. Old bridges with pre-tensioned edge beams have showed good result when it comes to the durability of the edge beam. 

Further investigations regarding the codes and regulations could be performed. By comparing codes in Sweden and Denmark an evaluation can be made of how much degrees of freedom that should be left for the engineers. 

In this master thesis a FEM-program was not used, only hand calculations were done. The use of a FEM program would give the possibility to do more global investigations of the edge beam. 

In this master thesis it was stated that the crack width for an integrated edge beam will exceed the limit over supports. It would be interesting to know how well the pre-fabricated and without a real edge beam would work.
8 Bibliography

Literature


BSSA, 2013-02-01. The use of stainless steel reinforcement in bridges. Arup research & development


Maglica, Adriano, 2012. Ölandsbrons kantbalkar har fått katodiskt skydd. Nr 1 Husbyggaren


Newton, Nigel, 2010. ”The use of semi – structured interviews”.


SVBRF. http://www.svbrf.se. 2014-02-01. (Svenska väg- och broräckesföreningen)


**Interviews**

Broman, Mats. Architect Swedish Transport Administration. 2014-04-08

Jyttner, Hans. Head of production Skanska. 2014-05-14


---

50
Pettersson, Anders, Location Manager NCC Construction. 2014-05-21
Pettersson, Lars. Professor at KTH – Royal Institute of Technology, 2014
Ronnebrant, Robert. Bridge specialist Trafikverket. 2014-04-14
Sandelius, Ulf. Bridge planner Broteknik Ulf Sandelius AB. 2014
Westberg, Niclas. Head of production Skanska. 2014-03-15

Pictures
Edge beam group, 2014.
NCC. http://www.ncc.se/. 2014-01-25
Appendix A: Questionnaire

KTH School of Architecture and the Built Environment
Division of Structural Engineering and Bridges

Design methods of concrete bridge slabs. Swedish and international study.
*(Dimensioneringsmetoder för brobaneplattor både i Sverige och internationellt)*

A typical edge beam is exposed to airborne pollutions (1) and water with chloride content (2).

Figure 38  The exposed edge beam (Fasheyi 2013)
General information

The main goal of this Master Thesis is to get insight into the question of why two different designers will often give different solutions for the design of the bridge deck slab including the edge beam provided the same conditions. By reading through the codes/standards in different countries some understanding of how the decision making process during the design phase is has been acquired. Therefor to understand the involvement and influence of the decision maker(s) regarding the bridge deck slab and edge beam, meetings are planned to be held with companies both in Sweden where the work of this master thesis has its base, as well as internationally where companies are involved in different procurement methods and designs.

To understand why the edge beam has a certain final design information such as calculations, drawings and what kind of decisions being made during the designing process have to be considered. Questions as if the Bridge edge beam is considered when the structural resistance of the bridge deck is evaluated, how local effects of the load are accounted in the calculations and how the load is distributed along the bridge deck will probably not be fully answered with the questions in this formulary. Therefore, eventually drawings and calculations will be needed to have a good approach to answer the topic of this master thesis. Concerning publishing private information, this will of course be discussed during the meetings so that it is really clarified what can and cannot be published.

The figure below indicates that when talking about the concrete bridge deck only the top part of the deck is in focus i.e. not the bottom part whether it is of steel, concrete or some other material.

![Illustration of concrete bridge deck](image)

Figure 39  Illustration of concrete bridge deck (Faridoo et al 2011)

All the answers received by the companies has been summarized in this questionnaire that was used during the interview sessions. Even though this is not general information about how all companies in Sweden works it gives a good overview of how things work in Sweden.

Tendering process

A variety of procurement methods exists and often the use of a method differs from country to country. The companies offer different kind of services connected to the procurement methods.
1. Which types of procurement methods are your company most familiar with?

Example of procurement methods are:

Design-build = Totalentreprenad
Design-bid-build = Utförandeentreprenad
General contracting = Generalentreprenad
Divided contracting = Delad entreprenad
Construction management
Public-private-partnership (PPP) = Offentlig-privat-samverkan (OPS)
Engineer-procure-construct (EPC)

The most common types of procurement methods that are used today is Design-build and Design-bid-build.

2. Is it some kind of administration or other building company that are your main supplier of building projects?

Road administration
Building company
Other (state who)

For a majority of times it is the Swedish Transport Administration (STA) that provides the building projects, in some cases different municipality and other building companies can be the supplier of projects.

3. If you are responsible for the design of the edge beam describe scenarios as preliminary designs, calculation verifications and possible optimization?

At first they are given a document of motion with all the specific geometry of the bridge. The designer may if he/her wants to readjust some of the thicknesses, the amount of reinforcement etc. but things as spans and so on are already determined. Thereafter the designer sends in their solution and cost for it which in turn will be supervised by someone, most often STA or some consult.

Concrete bridge deck

4. The following diagram describes how calculations for RC (Reinforced Concrete) slabs have evolved. What is your opinion regarding the improved design approach to apply in the upcoming future?

The general picture of implementing non-linear analysis to edge beams is that it is about time and money. It simply don’t feel like a complex non-linear analysis is necessary for an bam, in this case the edge beam.
5. Is the edge beam included in the evaluation of the structural resistance of the bridge deck? If not where is the “cut-line” located?

No the edge beam is not a part of the bridges structural resistance. The cut line is placed 100 mm from the edge beam according to TRVK Bro 2011 D.2.2.1.2. The idea is that the bridge should be dimensioned to hold traffic loads when the edge beam is replaced.

6. If the edge beam is included in the structural resistance, is it included in a local- or global analysis?

The edge beam is mostly not included in global analysis of the bridge but could be included in some cases when global FEM calculations are made. However the edge beam should withstand an impact to the railing and should therefore be designed for a local impact load. The edge beam is also included in an transverse distribution of concentrated loads on the bridge deck but in those cases the cantilever without an edge beam is also controlled to fulfill the criteria’s in TRVK Bro 2011.
7. Does the design of the concrete bridge deck influence the design of the edge beam? If yes, in which way (e.g. reinforcement etc.)? !!!!!

Mostly not but in some cases the impact load can affect the thickness of the neck between the edge beam and bridge deck.

**Edge beam**

8. When the architect is involved does it leave space for adjustment when it comes to the design?

The overall feeling is that when the architect is involved in the design of the edge beam it is not possible to change the design. Discussion can in some cases be held if the design of the edge beam would complicate the constructing phase.

9. Do you feel that it is time and money consuming when the architect is involved in the design? In case you think so, why and how?

Not for designers but probably for entrepreneurs when the design complicates things at the construction site.

10. In case you are responsible for the design of the edge beam in a certain project, which codes/rules are used when designing the edge beam? Until which extent are there degrees of freedom and which one do you consider to be the most important?

**TRVK Bro 11, TRVR Bro 11 and the Eurocodes.**

11. Which types of edge beams is your company most familiar with? What advantages/disadvantages would you say that they have? Some examples of edge beams are (some of these are visible in the figures that follows question 8):

- Integrated concrete edge beam
- Pre-fabricated concrete edge beam
- Without an real edge beam (Railing attached to the side)
- Steel edge beam
- Other

The most common type of edge beam used in Sweden is the integrated concrete edge beam.

12. Who decide which type of railing that will be used for the edge beam?

The type of railing used are stated in the documents of motion, the owner and architect take that decision.

13. What type(s) of railing(s) are your company most familiar with?
Appendix A: Questionnaire

- Concrete parapet = Betong räcke
- Guard rail = W-profilräcke
- Pipe railing = Rör-profilräcke
- Bar railing = kohlswaräcke
- Other (state which)

The most common type of railing today is the Guard rail and the Pipe railing, but this changes with time.

In Sweden the integrated concrete edge beam are mostly used, in North America are known for using concrete railings and false edge beams and in European countries such as Germany the not-integrated edge beam (brückenkappa) are common.

14. Have your company ever considering using edge beams solutions that are common in other countries?

**Prefabicrted edge beam**

**Steel edge beam**
Appendix A: Questionnaire

Integrated edge beam

Without an real edge beam
Appendix B: Landskapsbron

1) 2592-14-400-409 (overview)
2) 2592-14-400-405 (superstructure)
3) 2592-14-400-401 (edge beam, curvature)
4) 2592-14-400-432 (longitudinal reinforcement in edge beam)
5) 2592-14-400-428 (reinforcement in bridge deck)
6) 2592-14-400-434 (joints in edge beam)
Appendix C: Rotebro E4

1) 140K-20-01 (elevation, overview)
2) 142K-23-65 (edge beam)
3) 143K-23-65 (edge beam)
4) 142K-2371 (edge beam)
Appendix D: Drawings design study

1) (Kista gård – Bridge deck)
2) 147-K23-M3 (Dynekilen – Bridge deck and edge beam)
3) 147-K23-N1 (Dynekilen – Reinforcement in bridge deck)
4) 147-K23-N2 (Dynekilen – Overview of longitudinal reinforcement in bridge deck)
5) 14-1729-1 (Dynekilen – Edge beam and cantilever)
APPENDIX D: Drawings design study

73
Appendix E: Calculations

1) (Excel file from Kista bridge for estimating the amount of reinforcement in bridge deck)

2) (Excel file for compression height of pre-fabricated edge beam)

3) (Dimensioning of edge beam)

4) (Crack width for 7-9ϕ16mm)

5) (Crack width for maximum amount of reinforcement)

6) (Anchorage of pre-fabricated edge beam)

7) (Pucher diagram – Distribution of concentrated loads)

8) (Moment over the support with and without the edge beam)
## Reinforcement according to SS-EN 1992-1-1:2005 7.3.4

### Top part of bridge deck for Kista bridge

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M^\text{brott}$</td>
<td>kNm</td>
<td>$-32000$</td>
<td>Moment in ultimate limit state</td>
</tr>
<tr>
<td>$N^\text{brott}$</td>
<td>kN</td>
<td></td>
<td>Axial force in ultimate limit state</td>
</tr>
<tr>
<td>$M^\text{kvasi.}$</td>
<td>kNm</td>
<td>$-19750$</td>
<td>Moment for crack width calculations</td>
</tr>
<tr>
<td>$N^\text{kvasi.}$</td>
<td>kN</td>
<td></td>
<td>Axial force for crack width calculations</td>
</tr>
<tr>
<td>$H$</td>
<td>m</td>
<td>$1,744$</td>
<td>Total height of cross section</td>
</tr>
<tr>
<td>$B^\text{ök}$</td>
<td>m</td>
<td>$9,000$</td>
<td>Top part width of the cross section</td>
</tr>
<tr>
<td>$B^\text{uk}$</td>
<td>m</td>
<td>$2,800$</td>
<td>Bottom part width of the cross section</td>
</tr>
<tr>
<td>$c$</td>
<td>m</td>
<td>$0,060$</td>
<td>Concrete covering surface</td>
</tr>
<tr>
<td>$c/c\text{ lager}$</td>
<td>m</td>
<td>$0,070$</td>
<td>Vertical distance of reinforcement layers</td>
</tr>
<tr>
<td>Lager</td>
<td>pc</td>
<td>$2,0$</td>
<td>Number of reinforcement layers</td>
</tr>
<tr>
<td>$\phi$</td>
<td>mm</td>
<td>$25$</td>
<td>Diameter of reinforcement</td>
</tr>
<tr>
<td>$f^\text{yk}$</td>
<td>MPa</td>
<td>$500$</td>
<td>Characteristic strength of reinforcement</td>
</tr>
<tr>
<td>$E_s$</td>
<td>GPa</td>
<td>$200$</td>
<td>Modulus of elasticity of steel</td>
</tr>
<tr>
<td>$f^\text{ck}$</td>
<td>MPa</td>
<td>$30,0$</td>
<td>Characteristic tension capacity of concrete</td>
</tr>
<tr>
<td>$f^\text{ctm}$</td>
<td>MPa</td>
<td>$2,9$</td>
<td>Mean tensional capacity of concrete</td>
</tr>
<tr>
<td>$E_{cm}$</td>
<td>GPa</td>
<td>$33$</td>
<td>Characteristic modulus of elasticity for concrete</td>
</tr>
<tr>
<td>$\gamma_c$</td>
<td></td>
<td>$1,5$</td>
<td>Concrete factor</td>
</tr>
<tr>
<td>$\gamma_S$</td>
<td></td>
<td>$1,15$</td>
<td>Steel factor</td>
</tr>
<tr>
<td>$A^\text{brott}_s$</td>
<td>cm$^2$</td>
<td>$512$</td>
<td>Required amount of reinforcement in ultimate limit state</td>
</tr>
<tr>
<td>$A^\text{bruk}_s$</td>
<td>cm$^2$</td>
<td>$843$</td>
<td>Required amount of reinforcement for crack width control</td>
</tr>
<tr>
<td>Antal järn</td>
<td>st</td>
<td>$172$</td>
<td>Total number of reinforcement rebars</td>
</tr>
<tr>
<td>$c/c$</td>
<td>mm</td>
<td>$105$</td>
<td>Distance between the rebars</td>
</tr>
</tbody>
</table>
### APPENDIX E: Calculations

| \( w_k(\text{armtp}) \) | mm | 0.200 | Crack width at center of gravity for reinforcement |
| \( w_k(\text{ytan}) \) | mm | 0.230 | Crack width at concrete surface |

**Partial results**

| \( f_{cd} \) | MPa | 20.0 |
| \( f_{cd} \) | MPa | 1.93 |
| \( f_{yd} \) | MPa | 435 |
| \( \alpha_e \) | | 6.061 |
| \( d \) | m | 1.637 | Effective height |
| \( e \ (\text{armtp}) \) | m | 0.035 | Eccentricity from rebar to center of gravity of rebar group |
| \( A_{c,\text{eff}} \) | m² | 2.419 |
| \( \rho_{p,\text{eff}} \) | | 0.035 | \( \frac{A_s}{A_{c,\text{eff}}} \) |
| \( m_{\text{sid}}^{\text{kvasi}} \) | MNm | 19,750 |
| \( x^{\text{kvasi}} \) | m | 0.899 | Height of compression part |
| \( \sigma_s^{\text{kvasi}} \) | MPa | 175 | Design stress |
| \( s_{r,\text{max}} \) | mm | 297 | Maximum crack spacing |
| \( \varepsilon_{sm} - \varepsilon_{cm} \) | % | 0.675 |
| \( \alpha_{cc} \) | | 1.0 |
| \( \alpha_{ct} \) | | 1.0 |
| \( k_t \) | | 0.400 |
| \( k_1 \) | | 0.800 |
| \( k_2 \) | | 0.500 |
| \( k_3 \) | | 2.917 |
| \( k_4 \) | | 0.425 |
| \( m_{\text{sid}}^{\text{brott}} \) | MNm | 32,000 |
| \( m \) | | 0.2134 |
| $\Omega_s$ | 0.2429 |
## Height of compression part in pre-fabricated edge beam

<table>
<thead>
<tr>
<th>n</th>
<th>2</th>
<th>Number of M24 anchoring bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>c.c (m)</td>
<td>0,5</td>
<td>Concrete covering surface</td>
</tr>
<tr>
<td>$\sigma_{M24}$ (Pa)</td>
<td>800000000</td>
<td>Strength of one M24 bolt</td>
</tr>
<tr>
<td>$A_{M24}$</td>
<td>0,000353</td>
<td>Area M24 bolt</td>
</tr>
<tr>
<td>$k_2$</td>
<td>0,9</td>
<td></td>
</tr>
<tr>
<td>$\gamma_{M2}$</td>
<td>1,25</td>
<td>Partial factor for M24 bolt</td>
</tr>
<tr>
<td>$F_{M24}$ (N)</td>
<td>406656</td>
<td>Tensational load capacity of both anchorages</td>
</tr>
<tr>
<td>$F_{R.d}$ (N)</td>
<td>83190</td>
<td>Horizontal impact load</td>
</tr>
<tr>
<td>$F_H$ (N)</td>
<td>83190</td>
<td>Horizontal load at anchorage level</td>
</tr>
</tbody>
</table>

## Estimated force acting on the anchoring

<p>| $e_c$ (m) | 0,55 | Height from load to the covering surface of the bridge deck |
| $h_c$ (m) | 0,1 | Height of covering surface of the bridge deck |
| $\varphi_{anch}$ (m) | 0,024 | Diameter of anchorage bolt |
| $\varphi_{bd}$ (m) | 0,016 | Diameter of reinforcement rebar in bridge deck |
| $c_{bd}$ (m) | 0,045 | Concrete cover in the bridge deck |
| $M_1$ (Nm) | 60146,37 | Moment in the pre-fabricated edge beam |
| $E_\sigma$ (Pa) | 2E+11 | Elasticity of modulus for steel |
| $c_{eb.bd}$ (m) | 0,02 | Connection between edge beam and bridge deck |
| $H_{bd}$ (m) | 0,3 | Height of the bridge deck |
| $h_{cc}$ (m) | 0,02472107 | Height of the compression part |
| d | 0,227 | The effective height |
| $z_1$ (m) | 0,194639465 | The inner level arm |</p>
<table>
<thead>
<tr>
<th><strong>Symbol</strong></th>
<th><strong>Value</strong></th>
<th><strong>Description</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_c ) (N)</td>
<td>309014.2587</td>
<td>Force acting on anchorage coming from the moment</td>
</tr>
<tr>
<td>( F_{\text{tot}} ) (N)</td>
<td>392204.2587</td>
<td>Total force acting on anchorage</td>
</tr>
<tr>
<td>( f_{cc} ) (Pa)</td>
<td>30000000</td>
<td>Characteristic concrete compression strength</td>
</tr>
<tr>
<td>( \gamma_c )</td>
<td>1.2</td>
<td>Accidental factor for concrete</td>
</tr>
<tr>
<td>( f_{cd} ) (Pa)</td>
<td>25000000</td>
<td>Design concrete compression strength</td>
</tr>
<tr>
<td>( A_{cc} ) (m(^2))</td>
<td>0.01236057</td>
<td>Area of compression part</td>
</tr>
<tr>
<td>( h_{cc.d} ) (m)</td>
<td>0.024721141</td>
<td>New height of compression part</td>
</tr>
<tr>
<td>( \varepsilon_{\text{cu}} )</td>
<td>0.0035</td>
<td>Ultimate strain for concrete</td>
</tr>
<tr>
<td>( \varepsilon_s )</td>
<td>0.022210788</td>
<td>Strain in reinforcement</td>
</tr>
<tr>
<td>( \sigma_s )</td>
<td>4442157538</td>
<td>Design stress for reinforcement</td>
</tr>
</tbody>
</table>
Dimensioning of edge beam

Concrete class

\[ \gamma_s = 1.0 \]

Carrying capacity for reinforcement

\[ \gamma_c = 1.2 \]

Carrying capacity for concrete

\[ w_{eb} = 0.40 \text{ m} \]

Width of edge beam

\[ h_{eb} = 0.40 \text{ m} \]

Height of edge beam

\[ c_{cc} = 1.80 \text{ m} \]

Distance between railings

\[ h_{bd} = 0.170 \text{ m} \]

Height of bridge deck

\[ e_h = 0.55 \text{ m} \]

Eccentricity of point of attack (guard rail) to the top of the coating.

\[ l_{b,b} = 300 \text{ mm} \]

Length of bolt drilled into the edge beam

\[ \alpha := 2 \]

The edge beam should be dimensioned for a load twice as big as the load capacity of the railing

\[ t_{eb} := 100 \text{ mm} \]

height from surfacing to edge beam top

\[ n := 4 \]

n = number of bolts for railing

\[ a := 130 \text{ mm} \]

a = internal level arm between bolts

The load during an impact is distributed between two railings. The height/eccentricity of the load is seated at the level of the guard rail (navblijare). To get the dimensioning load for the edge beam the capacity of the railing, threaded bolt and anchoring of the bolt group is checked:

Failure in the threaded bolt

\[ A_s := 3.33 \text{ cm}^2 \]

\[ f_{yk,b} := 450 \times 10^6 \text{ Pa} \]

\[ \phi_{bolt} := 24 \text{ mm} \]

\[ F_v = A_s \cdot \frac{f_{yk,b}}{\gamma_s} \]

\[ M_{bar} := n \cdot F_v \cdot \frac{a}{2} \]

\[ e_{bolt} := e_h - t_{eb} \]

Diameter of bolt

Aₚ = nominell stress area

f民众 = yield limit of a bolt

F_v = characteristic tension force capacity of a bolt

M_bar = moment capacity of bolt group

e_bolt = eccentricity from load to top of edge beam
Section 3: Calculations

\[ F_{H,\text{bar}} = \frac{M_{\text{bar}}}{e_{\text{bolt}}} \]

\[ F_{H,\text{bar}} = \text{characteristic load on edge beam due to tension failure on the bolts} \]

\[ F_{H,\text{bar1}} = \alpha \cdot F_{H,\text{bar}} = 1.836 \times 10^5 \text{ N} \]

\[ F_{H,\text{bar1}} = \text{dimensioning load on the edge beam due to yield failure in bolt group} \]

**Anchoring failure of bolt** SS-EN 1992-1.1 8.4.2

\[ f_{\text{ctd}} = 4.20 \times 10^6 \text{ Pa} \]

\[ \eta_1 = 1.0 \quad \eta_2 = 1.0 \]

\[ f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{\text{ctd}} \]

\[ \sigma_{sd,b} = \frac{l_{b,b} \cdot b \cdot f_{bd}}{\phi_{\text{bolt}}} \]

**The design stress of the bar at the position from where the anchorage is measured from according to ss-en 92.1.1 8.4.3**

\[ F_{v,\text{anch}} = A_s \cdot \sigma_{sd,b} \]

\[ F_{\text{anch}} = n \cdot F_{v,\text{anch}} \frac{a}{2} \]

\[ M_{\text{anch}} = \frac{F_{\text{anch}}}{e_{\text{bolt}}} \]

\[ F_{H,\text{anch1}} = \alpha \cdot F_{H,\text{anch}} = 1.927 \times 10^5 \text{ N} \]

**Failure in the raling**

\[ f_{uk,r} = 235 \text{ MPa} \quad f_{ud} = \frac{f_{uk,r}}{\gamma_s} = 235 \text{ MPa} \]

\[ l_{\text{side}} = 55 \text{ mm} \]

\[ M_r < z \cdot f_{uk} \]

\[ z = \frac{l_{\text{side}}^3}{4} \]

\[ M_r = z \cdot f_{ud} \]

\[ f_{uk} = \text{characteristic yield limit of raling} \]

\[ l_{\text{side}} = \text{side length of an square section raling} \]

\[ z = \text{plastic bending resistance (m}^4\text{)} \]

\[ M_r = \text{Moment capacity of raling} \]
APPENDIX E: Calculations

\[ e_{HR} = e_{\text{bolt}} \]
\[ F_{HR} = \frac{M_r}{e_{HR}} \]
\[ M_s := F_{HR} \cdot e_{HR} \]
\[ F_{HR1} = \alpha \cdot F_{HR} = 4.344 \times 10^4 \text{N} \]
\[ F_{HR} = \text{dimensioning load capacity of the edge beam} \]

**Bending in edge beam**

The lowest load capacity will be the dimensioning one for the edge beam and will be used when the bending resistance of the edge beam is calculated.

\[ F_H := F_{HR1} \]
\[ f_{s,ym} := 500 \cdot 10^6 \text{Pa} \]
\[ f_{s,yd} := \frac{f_{s,ym}}{\gamma_s} = 5 \times 10^6 \text{Pa} \]
\[ n_{ed} := \frac{F_H}{c_c} \]
\[ e_m := e_{\text{bolt}} + \frac{h_{eb}}{2} = 0.65 \text{m} \]
\[ m_{bd} := \frac{F_H \cdot e_m}{c_c} \]
\[ c_{bd} := 45\text{mm} \]
\[ d = h_{eb} - c_{bd} \]

\[ A_{s,b} = \frac{m_{ed}}{0.9-d} + \frac{n_{ed}}{2} \cdot \frac{1}{f_{s,yd}} \]

The normal force are divided between the two railings and the moment are divided on the thickness of the section.

Often you have two sections, one for the edge beam and one for the edge beam. The difference between them will be the e-value and the d-value which in turn will affect the dimensioning moment as well as the required amount of reinforcement.

85
Calculating the degree of capacity utilization and the anchorage length

\[ A_{s,\text{reqd}} := \text{ } \]

\[ c_{c,r} := \text{ } \]

\[ A_{s,\text{used}} := \frac{1}{c_{c,r}} \cdot A_{\phi} \]

\[ n := \frac{A_{s,\text{reqd}}}{A_{s,\text{used}}} \]

\[ \sigma_{s,d,\text{b}} := \frac{f_{s,y,k}}{\gamma_{s}} \cdot n \]

\[ l_{bd} := \alpha_{1} \cdot \alpha_{2} \cdot \alpha_{3} \cdot \alpha_{4} \cdot \alpha_{5} \cdot l_{b,\text{reqd}} \]

**Calculations**

\[ A_{s,\text{reqd}} = \text{needed reinforcement due to bending and normal force, calculated using FEM or as in previous section} \]

\[ c_{c,r} = \text{distance between cross sectional reinforcement} \]

\[ A_{s,\text{used}} = \text{used reinforcement on a one meter strip with distance} \ c_{c,r} \ \text{between the reinforcement. Check TRVR D.1.4.1.6} \]

\[ n \text{ is the degree of capacity utilization} \]

\[ \sigma_{s,d,\text{b}} \text{ is the design stress of the bar (reinforcement) at the position from where the anchorage is measured from} \]

\[ l_{bd} = \text{design anchorage length. The} \ \alpha_{\text{-values can be determined with the help of table 8.2 in ss-en 92.1.1} \]

Shear and torsional control according to SS-EN 1992.1.1 (6.3.2)

\[ e_{T} := e_{m} \]

\[ T_{ed} := \frac{F_{H}}{2} \cdot e_{T} \]

\[ A_{eb} := h_{eb} \cdot w_{eb} \]

\[ u_{eb} := 2 \left( h_{eb} + w_{eb} \right) \]

\[ C_{c} := \alpha_{bd} \]

\[ \phi_{\text{shackle}} = 12 \text{mm} \]

\[ \phi_{\text{torsion}} = 16 \text{mm} \]

\[ t_{ef1} := 2 \left( C_{c} + \frac{\phi_{\text{shackle}}}{2} + \frac{\phi_{\text{torsion}}}{2} \right) \]

\[ t_{ef2} := \frac{A_{eb}}{u_{eb}} \]

\[ t_{ef} := \max (t_{ef1} \cdot t_{ef2}) \]

\[ e_{T} = \text{Eccentricity from point lead to the middle of the edge beam} \]

\[ T_{ed} = \text{Dimensioning torsional moment} \]

\[ A_{eb} = \text{Area of the edge beam} \]

\[ u_{eb} = \text{Outer circumference of the cross section} \]

\[ C_{c} = \text{Concrete cover chosen according to D.1.3.3 in TRVK/IR} \]

\[ \phi_{\text{shackle}} = \text{Diameter for shackle reinforcement} \]

\[ \phi_{\text{torsion}} = \text{Longitudinal torsion reinforcement} \]

\[ t_{ef1} = \text{The effective wall thickness maximum of} \ \frac{A_{u}}{u} \ \text{or twice the distance between the edge and center of longitudinal reinforcement.} \]
APPENDIX E: Calculations

\( A_r := (h_{eb} - \frac{t_{ef}}{2})(w_{eb} - \frac{t_{ef}}{2}) \)

The area enclosed by the center lines of the connecting walls, including hollow inner areas

\( u_k := 2\left( h_{eb} - \frac{t_{ef}}{2} \right) + 2\left( w_{eb} - \frac{t_{ef}}{2} \right) \)

The perimeter of the area \( A_k \)

\( 1 \leq \cot \theta \leq 2.5 \)

\( \cot \theta = 1.0 \%

\( A_{sl} := \frac{T_{ed} \cdot \cot \theta \cdot u_k}{2 \cdot A_k \cdot f_{s.yd}} \)

\( \text{ss-en 92.1.1} \)

\( 6.7 \text{N} \)

\( \theta = \text{the angle between the concrete compression strut and the beam axis perpendicular to the shear force} \)

\( z_1 := h_{eb} - t_{ef} \)

\( z_2 := w_{eb} - t_{ef} \)

The side length of a wall (i) defined by the distance between the intersection points with the adjacent walls

\( z_3 := \max(z_1, z_2) \)

equation 6.26 & 6.27 in ss-en 92.1.1 gives:

\( V_{Ed.T} := \frac{T_{ed}}{2 \cdot A_k} \cdot z_3 \)

Dimensioned shear force due to torsion

\( V_{Ed.F} := \frac{F_H}{2} \)

Dimensioned shear force

\( A_{s,v,Ed} := \frac{V_{Ed.T} + V_{Ed.F}}{0.9 \cdot d \cdot f_{s.yd} \cdot \cot \theta} \)

\( \text{ss-en 92.1.1} \)

\( 6.8 \text{ mm}^2 \)

Amount of reinforcement in the cross section (tvärgående armering) compare to TRVR D.1.4.1.6
Crack width check for edge beams over supports ss-en 92.1.1 (7.3.4)

\[ \phi_{eb} := 16\text{mm} \]  
Diameter of reinforcement rebar in edge beam

\[ \phi_{t, bd, eb} := 16\text{mm} \]  
Diameter of transverse reinforcement between bridge deck and edge beam

\[ \phi_{bygel} := 12\text{mm} \]  
Diameter for shackel reinforcement in edge beam

\[ \phi_{bd} := 25\text{mm} \]  
The bar diameter for bridge deck

\[ k_t := 0.4 \]  
For long term loading

\[ E_s := 200 \times 10^9\text{ Pa} \]  
Modulus of elasticity for reinforcement steel

\[ E_{cm} := 33 \times 10^9\text{ Pa} \]  
Mean modulus of elasticity for concrete

\[ B_{eb} := 450\text{mm} \]  
Width of edge beam

\[ \alpha_{cc} := 1.0 \]  
Coeff. long term effects of compressive strength according to ss-en 92.1.1 (3.1.6)

\[ \gamma_s := 1.15 \]  
Partial factors according to ss-en 92.1.1 (T2.1N)

\[ \gamma_c := 1.5 \]  

\[ f_{ck} := 30 \times 10^6\text{ Pa} \]  
Characteristic compression strength of concrete

\[ f_{cd} := \frac{\alpha_{cc} f_{ck}}{\gamma_c} = 2 \times 10^7\text{ Pa} \]  
Compressive concrete strength, design value

\[ \alpha_e := \frac{E_s}{E_{cm}} = 6.061 \]  
Mean value of the tensile strength of the concrete is \( f_{ctm} \) if cracks are expected after 28 days

\[ f_{ctm} := 2.9 \times 10^6\text{ Pa} \]  
The stress in the tension reinforcement assuming a cracked section

\[ \sigma_s := 175 \times 10^6\text{ Pa} \]  
Concrete cover edge beam

\[ c_{eb} := 55\text{mm} \]  
Concrete cover bridge deck

\[ c_{bd} := 45\text{mm} \]  
D.1.3.3

\[ H_{bd} := 1744\text{mm} \]  
Height bridge deck

\[ H_{eb} := 450\text{mm} \]  
Height of edge beam

\[ H_{eb, bd} := 200\text{mm} \]  
Height between top part of edge beam and bridge deck
\[ n_{bd} := 1 \quad \text{Rows of reinforcement in top part of bridge deck} \]
\[ n_{eb} := 1 \quad \text{Rows of reinforcement in top part of edge beam} \]

Point of pressure for edge beam and bridge deck for when the reinforcement are symmetric in the edge beam with three rebars on each row. From top to \( t_p \).

\[
\begin{align*}
\text{tp}_{bd1} := & \frac{2 \cdot c_{bd} + \phi_{bd}}{2} = 0.058 \text{ m} \quad \text{1 Row} \\
\text{tp}_{bd2} := & \frac{\pi \left( \frac{\phi_{bd}}{2} \right)^2 \left[ c_{bd} + \frac{\phi_{bd}}{2} \right] + (c_{bd} + 4 \phi_{bd})}{\pi \left( \frac{\phi_{bd}}{2} \right)^2} = 0.101 \text{ m} \\
\text{tp}_{eb1} := & \frac{\pi \left( \frac{\phi_{eb}}{2} \right)^2 n_{eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right)}{\pi \left( \frac{\phi_{eb}}{2} \right)^2 n_{eb}} = 0.063 \text{ m} \quad \text{1 Row} \\
\text{tp}_{eb2} := & \frac{\pi \left( \frac{\phi_{eb}}{2} \right)^2 n_{eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + \pi \left( \frac{\phi_{eb}}{2} \right)^2 n_{eb} \left( c_{eb} + 1.5 \phi_{eb} + 134 \text{ mm} \right)}{\pi \left( \frac{\phi_{eb}}{2} \right)^2 \left( n_{eb} + n_{eb} \right)} = 0.113 \text{ m} \\
\text{tp}_{eb3} := & \frac{\pi \left( \frac{\phi_{eb}}{2} \right)^2 n_{eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + n_{eb} \left( c_{eb} + 1.5 \phi_{eb} + 134 \text{ mm} \right) + n_{eb} \left( c_{eb} + 2.5 \phi_{eb} + 2.134 \text{ mm} \right)}{\pi \left( \frac{\phi_{eb}}{2} \right)^2 \left( n_{eb} + n_{eb} - n_{eb} \right)} = 0.196 \text{ m}
\end{align*}
\]
Section 3: Calculations

\[ t_{peb} := t_{peb3} \]
\[ t_{pbd} := t_{pbd2} \]
\[ d_1 := (H_{bd} + H_{eb, bd} - t_{peb}) - (H_{bd} - t_{pbd}) = 0.105 \text{ m} \] Distance from point of pressure in bridge deck to point of pressure in edge beam
\[ d := H_{bd} - t_{pbd} = 1.643 \text{ m} \] Distance from bottom of bridge deck to point of pressure in reinforcement of the bridge deck
\[ \varepsilon_{cu} := 0.0035 \] Ultimate compressive strain of concrete
\[ \varepsilon_{bd} := \frac{\sigma_s}{E_s} \] Strain in bridge deck

\[ \varepsilon_{eb} := \frac{\varepsilon_{bd} (d_1 + 0.9 \cdot d) + \varepsilon_{cu} \cdot d_1}{0.9 \cdot d} = 1.185 \times 10^{-3} \] Strain in edge beam (top part)

\[ \sigma_{s, eb} := \varepsilon_{eb} \cdot E_s = 2.371 \times 10^8 \text{ Pa} \]
\[ n_s := 9 \]

\[ A_{s, eb} := n_s \cdot \pi \left( \frac{\phi_{eb}}{2} \right)^2 = 1.81 \times 10^{-3} \text{ m}^2 \] Amount of reinforcement in top part of edge beam

\[ A_{c, eff, eb} := 2 \cdot t_{peb} \cdot E_{eb} = 0.177 \text{ m}^2 \]

\[ \rho_{p, eff, eb} := \frac{A_{s, eb}}{A_{c, eff, eb}} = 0.01 \]

\[ k_1 := 0.80 \quad k_2 := 0.50 \quad k_3 := 2.917 \quad k_4 := 0.425 \]

\[ s_{r, max, eb} := k_3 \cdot c_{eb} + \frac{k_1 \cdot k_2 \cdot k_4 \cdot \phi_{eb}}{\rho_{p, eff, eb}} = 0.426 \text{ m} \] The maximum cracks spacing

\[ w_{keb, max} := 0.150 \text{ mm} \] Maximum width of edge beam cracks ss-en 92.1.1 (7.3.1)

\[ w_{keb} := s_{r, max, eb} \left[ \frac{\sigma_{s, eb} - k_1 \cdot \frac{\varepsilon_{ctm}}{\rho_{p, eff, eb} \cdot (1 + \alpha_e \cdot \rho_{p, eff, eb})}}{E_s} \right] = 0.249 \text{ mm} \]
Crack width check for edge beams over supports for maximum amount of reinforcement ss-en 92.1.1 (7.3.4)

\( \phi_{eb} = 16\text{mm} \)  
Diameter of reinforcement rebar in edge beam

\( \phi_{tbd,eb} = 16\text{mm} \)  
Diameter of transverse reinforcement between bridge deck and edge beam

\( \phi_{bygel} = 12\text{mm} \)  
Diameter for shackle reinforcement in edge beam

\( \phi_{bd} = 25\text{mm} \)  
The bar diameter for bridge deck

\( k_t = 0.4 \)  
For long term loading

\( E_s = 200 \times 10^9\text{Pa} \)  
Modulus of elasticity for reinforcement steel

\( E_{cm} = 33 \times 10^9\text{Pa} \)  
Mean modulus of elasticity for concrete

\( B_{eb} = 450\text{mm} \)  
Width of edge beam

\( \alpha_c = 1.0 \)  
Coeff. long term effects of compressive strength according to ss-en 92.1.1 (3.1.6)

\( \gamma_s = 1.15 \)  
Partial factors according to ss-en 92.1.1 (T2.1N)

\( f_{ck} = 30 \times 10^6\text{Pa} \)  
Characteristic compression strength of concrete

\[ f_{cd} = \frac{\alpha_c f_{ck}}{\gamma_c} = 2 \times 10^7\text{Pa} \]  
Compressive concrete strength, design value

\[ \alpha_e = \frac{E_s}{E_{cm}} = 6.061 \]  
Mean value of the tensile strength of the concrete is \( f_{ctm} \) if cracks are expected after 28 days

\( f_{ctm} = 2.9 \times 10^6\text{Pa} \)

\( \sigma_s = 175 \times 10^6\text{Pa} \)

\( c_{eb} = 55\text{mm} \)  
Concrete cover edge beam

\( c_{bd} = 45\text{mm} \)  
Concrete cover bridge deck

\( H_{bd} = 1744\text{mm} \)  
Height bridge deck
Section 3: Calculations

$H_{eb} = 450\text{mm}$  
$H_{eb, bd} = 200\text{mm}$  

$\eta_{bd} = 1$  
$\eta_{eb} = 1$

Rows of reinforcement in top part of bridge deck  
Rows of reinforcement in top part of edge beam

Point of pressure for edge beam and bridge deck for when the reinforcement are symmetric in the edge beam

$$\bar{t}_{Pbd1} = \frac{2c_{bd} + \phi_{bd}}{2} = 0.058\text{m}$$

1 Row

$$\bar{t}_{Pbd2} = \frac{\left(\frac{\phi_{bd}}{2}\right)^2 \left[c_{bd} + \frac{c_{bd}}{2} + \frac{\phi_{bd}}{2} + 4\phi_{bd}\right]}{\pi \left(\frac{\phi_{bd}}{2}\right)^2 \cdot 2} = 0.101\text{m}$$

2 Row

The $\phi$ stands for the distance between the rebars vertically, from the bottom of one rebar to the top of the rebar beneath.

$$\phi_{16} = \frac{316\text{mm} - 4.16\text{mm}}{3} = 84.33\text{mm}$$

$$\phi_{20} = \frac{316\text{mm} - 5.20\text{mm}}{4} = 54.05\text{mm}$$

$$\phi_{25} = \frac{316\text{mm} - 4.25\text{mm}}{3} = 72.67\text{mm}$$

$$\phi_{32} = \frac{316\text{mm} - 3.32\text{mm}}{2} = 110.89\text{mm}$$

$\phi = \phi_{16}$

$n_{1, eb} = 4$  

Numbers of rebars on first level

$$\bar{t}_{Peb1} = \frac{\left(\frac{\phi_{eb}}{2}\right)^2 \left[c_{eb} + \frac{\phi_{eb}}{2}\right]}{\pi \left(\frac{\phi_{eb}}{2}\right)^2 \cdot n_{1, eb}} = 0.063\text{m}$$

1 Row
\[ n_{2,eb} = 4 \]
\[ \pi \left( \frac{\phi_{eb}}{2} \right)^2 n_{1,eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + \pi \left( \frac{\phi_{eb}}{2} \right)^2 n_{2,eb} \left( c_{eb} + 1.5 \phi_{eb} + \phi \right) \]
\[ = 0.113 \text{ m} \]

\[ n_{3,eb} = 4 \]
\[ \pi \left( \frac{\phi_{eb}}{2} \right)^2 \left[ n_{1,eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + n_{2,eb} \left( c_{eb} + 1.5 \phi_{eb} + \phi \right) + \ldots \right] \]
\[ + n_{3,eb} \left( c_{eb} + 2.5 \phi_{eb} + 2\phi \right) \]
\[ \pi \left( \frac{\phi_{eb}}{2} \right)^2 \left[ n_{1,eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + n_{2,eb} \left( c_{eb} + 1.5 \phi_{eb} + \phi \right) + \ldots \right] \]
\[ + n_{3,eb} \left( c_{eb} + 2.5 \phi_{eb} + 2\phi \right) \]
\[ + n_{4,eb} \left( c_{eb} + 3.5 \phi_{eb} + 3\phi \right) \]
\[ = 0.163 \text{ m} \]

\[ n_{4,eb} = 4 \]
\[ \pi \left( \frac{\phi_{eb}}{2} \right)^2 \left[ n_{1,eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + n_{2,eb} \left( c_{eb} + 1.5 \phi_{eb} + \phi \right) + \ldots \right] \]
\[ + n_{3,eb} \left( c_{eb} + 2.5 \phi_{eb} + 2\phi \right) \]
\[ + n_{4,eb} \left( c_{eb} + 3.5 \phi_{eb} + 3\phi \right) \]
\[ + n_{4,eb} \left( c_{eb} + 4.5 \phi_{eb} + 4\phi \right) \]
\[ \pi \left( \frac{\phi_{eb}}{2} \right)^2 \left[ n_{1,eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + n_{2,eb} \left( c_{eb} + 1.5 \phi_{eb} + \phi \right) + \ldots \right] \]
\[ + n_{3,eb} \left( c_{eb} + 2.5 \phi_{eb} + 2\phi \right) \]
\[ + n_{4,eb} \left( c_{eb} + 3.5 \phi_{eb} + 3\phi \right) \]
\[ + n_{5,eb} \left( c_{eb} + 4.5 \phi_{eb} + 4\phi \right) \]
\[ = 0.213 \text{ m} \]

\[ n_{5,eb} = 4 \]
\[ \pi \left( \frac{\phi_{eb}}{2} \right)^2 \left[ n_{1,eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + n_{2,eb} \left( c_{eb} + 1.5 \phi_{eb} + \phi \right) + \ldots \right] \]
\[ + n_{3,eb} \left( c_{eb} + 2.5 \phi_{eb} + 2\phi \right) \]
\[ + n_{4,eb} \left( c_{eb} + 3.5 \phi_{eb} + 3\phi \right) \]
\[ + n_{5,eb} \left( c_{eb} + 4.5 \phi_{eb} + 4\phi \right) \]
\[ \pi \left( \frac{\phi_{eb}}{2} \right)^2 \left[ n_{1,eb} \left( c_{eb} + \frac{\phi_{eb}}{2} \right) + n_{2,eb} \left( c_{eb} + 1.5 \phi_{eb} + \phi \right) + \ldots \right] \]
\[ + n_{3,eb} \left( c_{eb} + 2.5 \phi_{eb} + 2\phi \right) \]
\[ + n_{4,eb} \left( c_{eb} + 3.5 \phi_{eb} + 3\phi \right) \]
\[ + n_{5,eb} \left( c_{eb} + 4.5 \phi_{eb} + 4\phi \right) \]
\[ = 0.263 \text{ m} \]
Section 3: Calculations

\( n_{6,eb} := 4 \)

\[
\begin{align*}
\theta_{eb6} := & \frac{\pi \left( \frac{\phi_{eb}}{2} \right)^2}{\pi \left( \frac{\phi_{eb}}{2} \right)^2} \left[ n_{1,eb} (c_{eb} + \frac{\phi_{eb}}{2}) + n_{2,eb} (c_{eb} + 1.5 \phi_{eb} + \phi) + 
\right. \\
& \left. + n_{3,eb} (c_{eb} + 2.5 \phi_{eb} + 2\phi) + n_{4,eb} (c_{eb} + 3.5 \phi_{eb} + 3\phi) + 
\right. \\
& \left. + n_{5,eb} (c_{eb} + 4.5 \phi_{eb} + 4\phi) + n_{6,eb} (c_{eb} + 5.5 \phi_{eb} + 5\phi) \right] \\
= & 0.313 \text{ m}
\end{align*}
\]

\( \theta_{eb} = \theta_{eb6} \)

\( \theta_{bd} = \theta_{bd2} \)

\( d_1 := (H_{bd} + H_{eb,bd} - \theta_{eb}) - (H_{bd} - \theta_{bd}) = -0.012 \text{ m} \) Distance from point of pressure in bridge deck to point of pressure in edge beam

\( d := H_{bd} - \theta_{bd} = 1.643 \text{ m} \) Distance from bottom of bridge deck to point of pressure in reinforcement of the bridge deck

\( \varepsilon_{cu} := 0.0035 \) Ultimate compressive strain of concrete

\( \varepsilon_{bd} := \frac{\sigma_s}{E_s} \) Strain in bridge deck

\( \varepsilon_{eb} := \frac{\varepsilon_{bd} \cdot (d_1 + 0.9 \cdot d) + \varepsilon_{cu} \cdot d_1}{0.9 \cdot d} = 8.402 \times 10^{-4} \) Strain in edge beam (top part)

\( \sigma_{s,eb} := \varepsilon_{eb} \cdot E_s = 1.68 \times 10^8 \text{ Pa} \)

\( n_s = n_{1,eb} + n_{2,eb} + n_{3,eb} + n_{4,eb} + n_{5,eb} + n_{6,eb} = 24 \)

\( A_{3,eb} := n_s \cdot \pi \left( \frac{\phi_{eb}}{2} \right)^2 = 4.825 \times 10^{-3} \text{ m}^2 \) Amount of reinforcement in top part of edge beam

\( A_{c,effective,eb} := 2 \cdot \theta_{eb} \cdot B_{eb} = 0.282 \text{ m}^2 \)
\[ \rho_{p,\text{eff.eb}} := \frac{A_{s,eb}}{A_{c,\text{eff.eb}}} = 0.017 \]

\[ k_1 := 0.80 \quad k_2 := 0.50 \quad k_3 := 2.917 \quad k_4 := 0.425 \]

\[ S_{r,\text{max.eb}} = k_3 c_{eb} + \frac{k_1 k_2 k_4 \phi_{eb}}{\rho_{p,\text{eff.eb}}} = 0.319 \text{m} \]

The maximum cracks spacing

\[ w_{k,eb,\text{max}} = 0.150 \text{mm} \]

Maximum width of edge beam cracks ss-en 92.1.1 (7.3.1)

\[ w_{k,eb} = \frac{\sigma_{s,eb} \cdot \frac{f_{ctm}}{\rho_{p,\text{eff.eb}}} \cdot (1 + \alpha_e \cdot \rho_{p,\text{eff.eb}})}{E_s} = 0.149 \text{mm} \]
Section 3: Calculations

\[ W_{ed} = 450\text{mm} \quad H_{eb} = 450\text{mm} \quad H_{bd} = 250\text{mm} \]

Width of edge beam  
Height of edge beam  
Height of bridge deck

\[ \phi_{bd} = 16\text{mm} \]

Diameter of rebar in bridge deck

\[ \alpha := 2 \]

The edge beam is designed for a load twice as big as the load capacity of the railing system, according to en1991.2 (4.7.3.3.(2))

\[ \gamma_s := 1.0 \quad \gamma_c := 1.2 \]

Accidental Partial factors for steel and concrete en1992.1.1 (2.4.2.4)

\[ c_{bd} := 45\text{mm} \]

Concrete cover for edge beam

\[ c_{eb, bd} := 20\text{mm} \]

Connection between edge beam and bridge deck

\[ f_{uk} := 360 \times 10^6 \text{Pa} \]

Design yield limit of railing

\[ e_c := 550\text{mm} \]

Height from point of load to the covering surface of the bridge deck

\[ h_c := 100\text{mm} \]

Height of covering surface of the bridge deck

\[ h_{c, eb} := 90\text{mm} \]

Height from covering surface to the fotplate of railing

\[ h_1 = e_c - h_c - h_{c, eb} = 0.36\text{m} \]

Height between fotplate and point of load

\[ l_{side} = 55\text{mm} \]

The side length of the railing, assumed to be quadratic

\[ c_c := 500\text{mm} \]

Distance between the anchorage bolts of the pre fabricated edge beam

\[ \frac{3}{4} = 4.159 \times 10^4 \text{mm}^3 \]

Plastic bending resistance of the railing

**Moment capacity of the railing**

\[ M_R := \frac{3}{4} f_{uk} = 1.497 \times 10^4 \text{N} \cdot \text{m} \]

Bending resistance of the railing

\[ M.S = F.R \times h_{1} \]

Bending moment in fotplate from point load

\[ M.R = M.S \]

The bending resistance of the railing should be as big as the moment from the point load in the fotplate of the railing

\[ F_{R,k} := \frac{M.R}{h_{1}} = 4.159 \times 10^4 \text{N} \]

Characteristic load capacity of the railing
APPENDIX E: Calculations

\[ F_{R.d} = \alpha F_{R.k} = 8.319 \times 10^4 \text{N} \]

Design load capacity of railing

**Estimated force acting on the anchoring**

\[ F_H := F_{R.d} = 8.319 \times 10^4 \text{N} \]

Horizontal force acting in the anchorage of edge beam

\[ M_1 := F_{R.d} \left( c_c + h_c + c_{bd} + \phi_{bd} + 10.6 \text{mm} \right) = 60.028 \text{kN} \cdot \text{m} \]

Moment in the pre fabricated edge beam

\[ E_s := 200 \times 10^9 \text{Pa} \]

Modulus of elasticity for reinforcement steel

\[ E_{cm} = 33 \times 10^9 \text{Pa} \]

Mean modulus of elasticity for concrete

\[ \alpha_e = \frac{E_s}{E_{cm}} = 6.061 \]

\[ z_1 = h_{bd} - c_{bd} - 12 \text{mm} - \phi_{bd} - c_{eb.bd} - 10 \text{mm} = 147 \text{mm} \]

\[ F_c := \frac{M_1}{z_1} = 408.354 \text{kN} \]

Concrete compression force

\[ f_{ck} := 30 \times 10^6 \text{Pa} \]

Concrete compression strength

\[ f_{cd} := \frac{f_{ck}}{\gamma_c} = 2.5 \times 10^7 \text{Pa} \]

\[ A_{cc} := \frac{F_c}{f_{cd}} = 1.333 \times 10^4 \text{mm}^2 \]

Area of compression part

\[ h_{cc} := \frac{A_{cc}}{c_c} = 32.668 \text{mm} \]

Height of compression part

\[ F_{tot} = F_H + F_c = 4.915 \times 10^5 \text{N} \]

**Strength of the anchoring bolts**

**M24 - Bolts, class 8.8**

\[ n := 2 \]

\[ \gamma_{M2} := 1.25 \]

\[ \sigma_{M2} := 800 \times 10^6 \text{Pa} \]

\[ A_{M2} := 353 \text{mm}^2 \]

\[ k_2 := 0.9 \]

\[ F_{1,Rd} := n k_2 \sigma_{M2} A_{M2} \gamma_{M2} = 4.067 \times 10^5 \text{N} \]

According to table 3.4 ss-en 93 1.8
\[ \frac{W}{\text{Nm}} = 9.86 \times 10^{-5} \times \frac{811}{\pi + 1.51 \times 10^{-2}} \]

(Chart 17: Support-moment influence surface for the rotational edge of a clamped plate edge (figure - 13.1.8) x - y plane)

(Chart 17: Support-moment influence for the rotational edge of a clamped plate edge in the horizontal plane (figure - 13.1.8) x - y plane)
Material parameters

\[ f_{ck} := 35 \cdot 10^6 \text{Pa} \]

Compressive concrete strength - characteristic

\[ f_{ck} := 2.2 \cdot 10^6 \text{Pa} \]

Tensile strength of concrete - characteristic

\[ f_{cm} := 3.2 \cdot 10^6 \text{Pa} \]

Mean value of the tensile strength of the concrete

\[ f_{yk} := 500 \cdot 10^6 \text{Pa} \]

Yield strength of reinforcement

Loads

\[ t_{sf} := 0.095 \text{m} \]

Thickness of surface

\[ \gamma_{sf} := 23.5 \cdot 10^3 \frac{\text{N}}{\text{m}^3} \]

Density of surfac

\[ q_{sf} := t_{sf} \cdot \gamma_{sf} = 2.232 \cdot 10^3 \frac{\text{N}}{\text{m}^2} \]

Surfacing load

\[ \gamma_{concrete} := 25 \cdot 10^3 \frac{\text{N}}{\text{m}^3} \]

Density of concrete

\[ F_1 := 1.35 \cdot 10^3 \text{N} \]

Load from vehicle

Calculation of t.p and moment of inertia

\[ h_{eb} := 465 \text{mm} \]

Height of edge beam

\[ w_{eb} := 450 \text{mm} \]

Width of edge beam

\[ l_{fl} := 500 \text{mm} \]

Length to load F1

\[ h_{bd,free} := 250 \text{mm} \]

Height of bridge deck close to the edge beam

\[ h_{bd,sup} := 280 \text{mm} \]

Height of bridge deck over the support

\[ h_b := 256.5 \text{mm} \]

Height of bridge deck under the point load

\[ L_s := 2250 \text{mm} \]

Length of slab

\[ L_{fl} := 1750 \text{mm} \]

Length from load to support
APPENDIX E: Calculations

According to old standards

\[
t_{\text{pe}b} = \frac{h_{eb} \cdot w_{eb} \cdot \frac{h_{eb}}{2} + l_1 \cdot \frac{(h_{bd,free} + h_b)^2}{8}}{h_{eb} \cdot w_{eb} + l_1 \cdot \frac{(h_{bd,free} + h_b)}{2}} = 0.193 \text{ m}
\]

point of pressure for edge beam part

\[
\frac{256.5 + 250}{2} = 253.25
\]

Bending in the longitudinal direction

\[
I_{eb} := h_{eb} \cdot w_{eb} \left( t_{\text{pe}b} - \frac{h_{eb}}{2} \right)^2 + w_{eb} \cdot h_{eb}^3 \frac{12}{12} + l_1 \cdot 253.25 \text{mm} \left( t_{\text{pe}b} - \frac{253.25 \text{mm}}{2} \right)^2 = 5.331 \times 10^6 \text{mm}^4
\]

\[
h_{bd,free} \div h_{bd,sup} = 0.893 \quad k := \frac{1}{1.05} = 0.952
\]

Bending in the transverse direction

\[
i_{\text{sup}} := \frac{h_{bd,sup}^3}{12} = 1.829 \times 10^{-3} \frac{m^4}{m}
\]

moment of inertia per meter at the support

\[
i_{\text{free}} := \frac{h_{bd,free}^3}{12} = 1.302 \times 10^{-3} \frac{m^4}{m}
\]

\[
m_{1,\text{max}} := F_1 \cdot \frac{h_{bd,\text{sup}}}{4} \left( \frac{k \cdot l_1}{h_{bd,\text{sup}} \cdot I_{eb}} \right)^{0.25} = 5.453 \times 10^4 \frac{N \cdot m}{m}
\]

\[
m_f := \frac{h_{bd,\text{sup}}}{4} \left( \frac{k \cdot l_1}{h_{bd,\text{sup}} \cdot I_{eb}} \right)^{0.25} = 0.405
\]

\[
\lambda_1 := \frac{m_f}{L_1} = 2 = 0.462 \frac{1}{m}
\]

b_b := 1.2 m

Boogiedistance

\[
m_{2,\text{max}} := 2 \cdot \frac{2}{\lambda_1} \cdot m_{1,\text{max}} = 8.552 \times 10^4 \frac{N \cdot m}{m}
\]
According to Pucher

When the surface of the tires are on each side of the middle

\[ M_{p,1} = \frac{2 \cdot 9.2}{\pi \cdot 8} \cdot F_1 = 9.884 \times 10^4 \frac{N \cdot m}{m} \]

When the surface of the tires starts from the middle

\[ M_{p,2} = \frac{10.4 + 7}{\pi \cdot 8} \cdot F_1 = 9.346 \times 10^4 \frac{N \cdot m}{m} \]

Difference

\[ \text{diff} := \frac{M_{p,1}}{m_{2, \text{max}}} - 1.156 \]

15.8 percent higher moment when the edge beam is not included

Loads

Distributed load

\[ \gamma_d = 6.3 \cdot 10^3 \frac{N}{m^2} \quad q_d := \gamma_d \cdot L_s = 1.417 \times 10^4 \frac{N}{m} \quad x_d = 875 \text{mm} \]

\[ M_d := q_d \cdot x_d = 1.24 \times 10^4 \frac{N \cdot m}{m} \]

Edge beam

\[ q_{eb} := h_{eb} \cdot \gamma_{eb} \cdot \gamma_{\text{concrete}} = 5.231 \times 10^3 \frac{N}{m} \quad x_{eb} = 2475 \text{mm} \]

\[ M_{eb} := q_{eb} \cdot x_{eb} = 1.295 \times 10^4 \frac{N \cdot m}{m} \]

Concrete slab

\[ q_{cs} = \left( \frac{h_{bd, \text{free}} + h_{bd, \text{sup}}}{2} \right) \cdot \gamma_{\text{concrete}} = 1.491 \times 10^4 \frac{N}{m} \quad x_{bd} = 1125 \text{mm} \]

\[ M_{cs} := q_{cs} \cdot x_{bd} = 1.677 \times 10^4 \frac{N \cdot m}{m} \]

Railing

\[ q_r = 0.40 \cdot 10^3 \frac{N}{m} \quad x_r = 2325 \text{mm} \]

\[ M_r := q_r \cdot x_r = 930 \frac{N \cdot m}{m} \]

\[ \gamma_p = 1.1 \]

Pavement

\[ q_p := q_{sf} \cdot L_s \cdot \gamma_p = 5.525 \times 10^3 \frac{N}{m} \quad x_p = x_{bd} \]

\[ M_p := q_p \cdot x_p = 6.216 \times 10^4 \frac{N \cdot m}{m} \]
APPENDIX E: Calculations

\[ M_{uls} \]

\[ M_{uls.\, eb} = 1.2 (M_{eb} + M_{cs} + M_{r}) + 1.32 M_p + 1.5 (M_{p,1} + M_d) = 2.118 \times 10^5 \frac{N\cdot m}{m} \]

with edge beam

\[ M_{uls} = 1.2 (M_{cs} + M_{r}) + 1.32 M_p + 1.5 (m_{2,\, max} + M_d) = 1.763 \times 10^5 \frac{N\cdot m}{m} \]

without edge beam

\[ M_{sls} \]

\[ M_{sls.\, eb} = 1.0 (M_{eb} + M_{cs} + M_{r}) + 1.0 M_p + 0.5 (M_{p,1} + M_d) = 3.686 \times 10^4 \frac{N\cdot m}{m} \]

with edge beam

\[ M_{sls} = 1.0 (M_{cs} + M_{r}) + 1.0 M_p + 0.5 (m_{2,\, max} + M_d) = 2.392 \times 10^4 \frac{N\cdot m}{m} \]

without edge beam

**Amount of reinforcement from excel file**

\( \phi_{bd} = 20\, \text{mm} \)

**With edge beam**

\[ A_{s.\, uls.\, eb} = 25\, \text{cm}^2 \]

Amount of reinforcement when failure

\[ A_{s.\, sls.\, eb} = 10.2\, \text{cm}^2 \]

Amount of reinforcement for crack width control

\( n_1 = 8 \)

Numbers of rebar’s

**Without edge beam**

\[ A_{s.\, uls} = 20\, \text{cm}^2 \]

Amount of reinforcement when failure

\[ A_{s.\, sls} = 7.9\, \text{cm}^2 \]

Amount of reinforcement for crack width control

\( n_2 = 7 \)

Numbers of rebar’s

\[ \frac{A_{s.\, sls.\, eb}}{A_{s.\, sls}} = 1.291 \]